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8.1 Overview

This chapter covers the geotechnical design of bridge foundations, cut-and-cover tunnel foundations, foundations for walls, and hydraulic structure foundations (pipe arches, box culverts, flexible culverts, etc.). **WSDOT GDM Chapter 17** covers foundation design for lightly loaded structures, and **WSDOT GDM Chapter 18** covers foundation design for marine structures. Both shallow (e.g., spread footings) and deep (piles, shafts, micro-piles, etc.) foundations are addressed. In general, the load and resistance factor design approach (LRFD) as prescribed in the AASHTO LRFD Bridge Design Specifications shall be used, unless a LRFD design methodology is not available for the specific foundation type being considered (e.g., micro-piles). Structural design of bridge and other structure foundations is addressed in the WSDOT LRFD Bridge Design Manual (BDM).

8.2 Overall Design Process for Structure Foundations

The overall process for geotechnical design is addressed in **WSDOT GDM Chapters 1 and 23**. For design of structure foundations, the overall WSDOT design process, including both the geotechnical and structural design functions, is as illustrated in Figure 8-1.

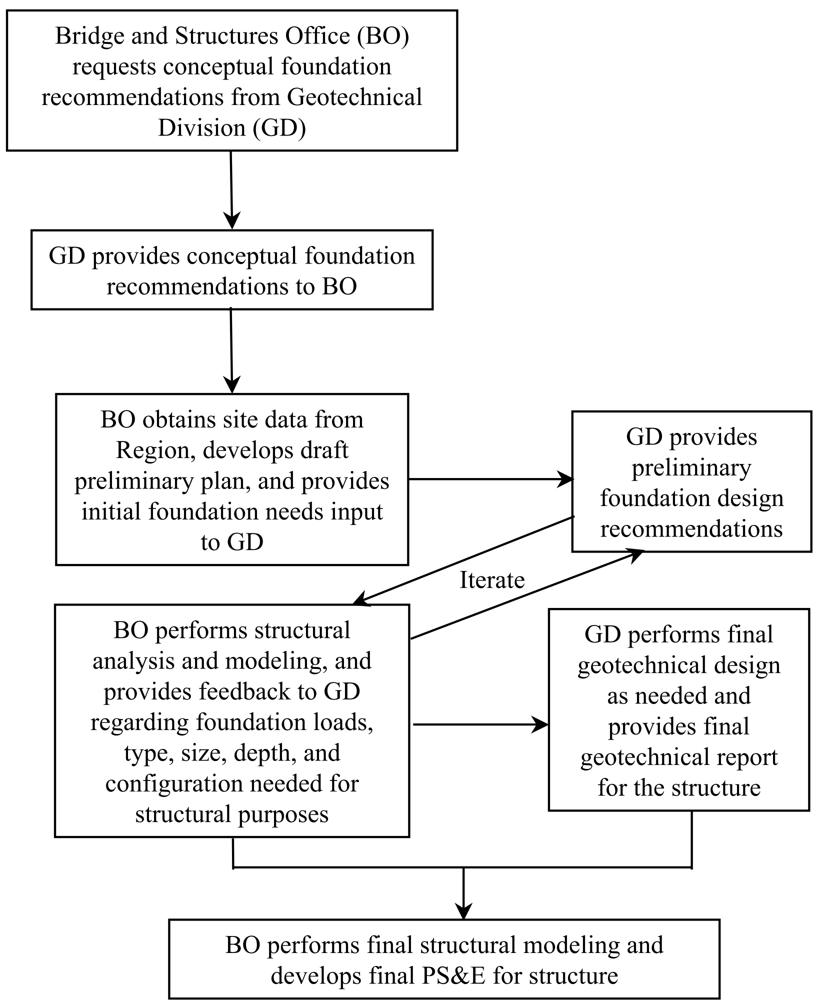


Figure 8-1 Overall design process for LRFD foundation design.

The steps in the flowchart are defined as follows:

Conceptual Bridge Foundation Design – This design step results in an informal communication/report produced by the Geotechnical Division at the request of the Bridge and Structures Office. This informal communication/report, consistent with what is described for conceptual level geotechnical reports in **WSDOT GDM Chapter 23**, provides a brief description of the anticipated site conditions, an estimate of the maximum slope feasible for the bridge approach fills for the purpose of determining bridge length, conceptual foundation types feasible, and conceptual evaluation of potential geotechnical hazards such as liquefaction. The purpose of these recommendations is to provide enough geotechnical information to allow the bridge preliminary plan to be produced. This type of conceptual evaluation could also be applied to other types of structures, such as tunnels or special design retaining walls.

Develop Site data and Preliminary Plan – During this phase, the Bridge and Structures Office obtains site data from the Region (see WSDOT Design Manual, Chapters 510, 1110, and 1130) and develops a preliminary bridge plan (or other structure) adequate for the Geotechnical Division to locate borings in preparation for the final design of the structure (i.e., pier locations are known with a relatively high degree of certainty). The Bridge and Structures Office would also provide the following information to the Geotechnical Division to allow them to adequately develop the preliminary foundation design:

- Anticipated structure type and magnitudes of settlement (both total and differential) the structure can tolerate.
- At abutments, the approximate maximum elevation feasible for the top of the foundation in consideration of the foundation depth.
- For interior piers, the number of columns anticipated, and if there will be single foundation elements for each column, or if one foundation element will support multiple columns.
- At stream crossings, the depth of scour anticipated, if known. Typically, the Geotechnical Division will pursue this issue with the HQ Hydraulics Office.
- Any known constraints that would affect the foundations in terms of type, location, or size, or any known constraints which would affect the assumptions which need to be made to determine the nominal resistance of the foundation (e.g., utilities that must remain, construction staging needs, excavation, shoring and falsework needs, other constructability issues).

Preliminary Foundation Design – This design step results in a memorandum produced by the Geotechnical Division at the request of the Bridge and Structures Office that provides geotechnical data adequate to do the structural analysis and modeling for all load groups to be considered for the structure. The geotechnical data is preliminary in that it is not in final form for publication and transmittal to potential bidders. In addition, the foundation recommendations are subject to change, depending on the results of the structural analysis and modeling and the effect that modeling and analysis has on foundation types, locations, sizes, and depths, as well as any design assumptions made by the geotechnical designer. Preliminary foundation recommendations may also be subject to change depending on the construction staging needs and other constructability issues that are discovered during this design phase. Geotechnical work conducted during this stage typically includes completion of the field exploration program to the final PS&E level, development of foundation types and capacities feasible, foundation depths needed, P-y curve data and soil spring data for seismic modeling, seismic site characterization and estimated ground acceleration, and recommendations to address known constructability issues. A description of subsurface conditions and a preliminary subsurface profile would also be provided at this stage, but detailed boring logs and laboratory test data would usually not be provided.

Structural Analysis and Modeling – In this phase, the Bridge and Structures Office uses the preliminary foundation design recommendations provided by the Geotechnical Division to perform the structural modeling of the foundation system and superstructure. Through this modeling, the Bridge and Structures Office determines and distributes the loads within the structure for all appropriate load cases, factors the loads as appropriate, and sizes the foundations using the foundation nominal resistances and resistance factors provided by the Geotechnical Division. Constructability and construction staging needs would continue to be investigated during this phase. The Bridge and Structures Office would also provide the following feedback to the Geotechnical Division to allow them to check their preliminary foundation design and produce the Final Geotechnical Report for the structure:

- Anticipated foundation loads (including load factors and load groups used).
- Foundation size/diameter and depth required to meet structural needs.
- Foundation details that could affect the geotechnical design of the foundations.
- Size and configuration of deep foundation groups.

Final Foundation Design - This design step results in a formal geotechnical report produced by the Geotechnical Division that provides final geotechnical recommendations for the subject structure. This report includes all geotechnical data obtained at the site, including final boring logs, subsurface profiles, and laboratory test data, all final foundation recommendations, and final constructability recommendations for the structure. At this time, the Geotechnical Division will check their preliminary foundation design in consideration of the structural foundation design results determined by the Bridge and Structures Office, and make modifications to the preliminary foundation design as needed to accommodate the structural design needs provided by the Bridge and Structures Office. It is possible that much of what was included in the preliminary foundation design memorandum may be copied into the final geotechnical report, if no design changes are needed. This report will also be used for publication and distribution to potential bidders.

Final Structural Modeling and PS&E Development – In this phase, the Bridge and Structures Office makes any adjustments needed to their structural model to accommodate any changes made to the geotechnical foundation recommendations as transmitted in the final geotechnical report. From this, the bridge design and final PS&E would be completed.

Note that a similar design process should be used if a consultant or design-builder is performing one or both design functions.

8.3 Data Needed for Foundation Design

The expected project requirements and subsurface conditions should be analyzed to determine the type and quantity of information to be developed during the geotechnical investigation. During this phase it is necessary to:

- Identify design and constructability requirements (e.g. provide grade separation, transfer loads from bridge superstructure, provide for dry excavation) and their effect on the geotechnical information needed
- Identify performance criteria (e.g. limiting settlements, right of way restrictions, proximity of adjacent structures) and schedule contraints
- Identify areas of concern on site and potential variability of local geology
- Develop likely sequence and phases of construction and their effect on the geotechnical information needed

- Identify engineering analyses to be performed (e.g. bearing capacity, settlement, global stability)
- Identify engineering properties and parameters required for these analyses
- Determine methods to obtain parameters and assess the validity of such methods for the material type and construction methods
- Determine the number of tests/samples needed and appropriate locations for them.

Table 8-1 provides a summary of information needs and testing considerations for foundation design.

Foundation Type	Engineering Evaluations	Required Information for Analyses	Field Testing	Laboratory Testing
Shallow Foundations	• bearing capacity • settlement (magnitude & rate) • shrink/swell of foundation soils (natural soils or embankment fill) • frost heave • scour (for water crossings) • liquefaction	• subsurface profile (soil, groundwater, rock) • shear strength parameters • compressibility parameters (including consolidation, shrink/swell potential, and elastic modulus) • frost depth • stress history (present and past vertical effective stresses) • depth of seasonal moisture change • unit weights • geologic mapping including orientation and characteristics of rock discontinuities	• SPT (granular soils) • CPT • PMT • dilatometer • rock coring (RQD) • plate load testing • geophysical testing	• 1-D Oedometer tests • soil/rock shear tests • grain size distribution • Atterberg Limits • specific gravity • moisture content • unit weight • organic content • collapse/swell potential tests • intact rock modulus • point load strength test

Driven Pile Foundations	• pile end-bearing • pile skin friction • settlement • down-drag on pile • lateral earth pressures • chemical compatibility of soil and pile • drivability • presence of boulders/ very hard layers • scour (for water crossings) • vibration/heave damage to nearby structures • liquefaction	• subsurface profile (soil, ground water, rock) • shear strength parameters • horizontal earth pressure coefficients • interface friction parameters (soil and pile) • compressibility parameters • chemical composition of soil/ rock (e.g., potential corrosion issues) • unit weights • presence of shrink/swell soils (limits skin friction) • geologic mapping including orientation and characteristics of rock discontinuities	• SPT (granular soils) • pile load test • CPT • PMT • vane shear test • dilatometer • piezometers • rock coring (RQD) • geophysical testing	• soil/rock shear tests • interface friction tests • grain size distribution • 1-D Oedometer tests • pH, resistivity tests • Atterberg Limits • specific gravity • organic content • moisture content • unit weight • collapse/swell potential tests • intact rock modulus • point load strength test
Drilled Shaft Foundations	• shaft end bearing • shaft skin friction • constructability • down-drag on shaft • quality of rock socket • lateral earth pressures • settlement (magnitude & rate) • groundwater seepage/ dewatering/ potential for caving • presence of boulders/ very hard layers • scour (for water crossings) • liquefaction	• subsurface profile (soil, ground water, rock) • shear strength parameters • interface shear strength friction parameters (soil and shaft) • compressibility parameters • horizontal earth pressure coefficients • chemical composition of soil/ rock • unit weights • permeability of water-bearing soils • presence of artesian conditions • presence of shrink/swell soils (limits skin friction) • geologic mapping including orientation and characteristics of rock discontinuities • degradation of soft rock in presence of water and/or air (e.g., rock sockets in shales)	• installation technique test shaft • shaft load test • vane shear test • CPT • SPT (granular soils) • PMT • dilatometer • piezometers • rock coring (RQD) • geophysical testing	• 1-D Oedometer • soil/rock shear tests • grain size distribution • interface friction tests • pH, resistivity tests • permeability tests • Atterberg Limits • specific gravity • moisture content • unit weight • organic content • collapse/swell potential tests • intact rock modulus • point load strength test • slake durability

Table 8-1 Summary of information needs and testing considerations (modified after Sabatini, et al., 2002).

WSDOT GDM Chapter 5 covers the requirements for how the results from the field investigation, the field testing, and the laboratory testing are to be used separately or in combination to establish properties for design. The specific test and field investigation requirements needed for foundation design are described in the following sections.

8.3.1 Field Exploration Requirements for Foundations

Subsurface explorations shall be performed to provide the information needed for the design and construction of foundations. The extent of exploration shall be based on variability in the subsurface conditions, structure type, and any project requirements that may affect the foundation design or construction. The exploration program should be extensive enough to reveal the nature and types of soil deposits and/or rock formations encountered, the engineering properties of the soils and/or rocks, the potential for liquefaction, and the ground water conditions. The exploration program should be sufficient to identify and delineate problematic subsurface conditions such as karstic formations, mined out areas, swelling/collapsing soils, existing fill or waste areas, etc.

Borings should be sufficient in number and depth to establish a reliable longitudinal and transverse substrata profile at areas of concern, such as at structure foundation locations, adjacent earthwork locations, and to investigate any adjacent geologic hazards that could affect the structure performance. Guidelines on the number and depth of borings are presented in **Table 8-2**. While engineering judgment will need to be applied by a licensed and experienced geotechnical professional to adapt the exploration program to the foundation types and depths needed and to the variability in the subsurface conditions observed, the intent of **Table 8-2** regarding the minimum level of exploration needed should be carried out. Geophysical testing may be used to guide the planning of the subsurface exploration and reduce the requirements for borings. The depth of borings indicated in **Table 8-2** performed before or during design should take into account the potential for changes in the type, size and depth of the planned foundation elements.

Table 8-2 shall be used as a starting point for determining the locations of borings. The final exploration program should be adjusted based on the variability of the anticipated subsurface conditions as well as the variability observed during the exploration program. If conditions are determined to be variable, the exploration program should be increased relative to the requirements in **Table 8-2** such that the objective of establishing a reliable longitudinal and transverse substrata profile is achieved. If conditions are observed to be homogeneous or otherwise are likely to have minimal impact on the foundation performance, and previous local geotechnical and construction experience has indicated that subsurface conditions are homogeneous or otherwise are likely to have minimal impact on the foundation performance, a reduced exploration program relative to what is specified in **Table 8-2** may be considered. Even the best and most detailed subsurface exploration programs may not identify every important subsurface problem condition if conditions are highly variable. The goal of the subsurface exploration program, however, is to reduce the risk of such problems to an acceptable minimum.

For situations where large diameter rock socketed shafts will be used or where drilled shafts are being installed in formations known to have large boulders, or voids such as in karstic or mined areas, it may be necessary to advance a boring at the location of each shaft.

In a laterally homogeneous area, drilling or advancing a large number of borings may be redundant, since each sample tested would exhibit similar engineering properties. Furthermore, in areas where soil or rock conditions are known to be very favorable to the construction and performance of the foundation type likely to be used (e.g., footings on very dense soil, and groundwater is deep enough to not be a factor), obtaining fewer borings than provided in **Table 8-2** may be justified. In all cases, it is necessary to understand how the design and construction of the geotechnical feature will be affected by the soil and/or rock mass conditions in order to optimize the exploration.

Samples of material encountered shall be taken and preserved for future reference and/or testing. Boring logs shall be prepared in detail sufficient to locate material strata, results of penetration tests, groundwater, any artesian conditions, and where samples were taken. Special attention shall be paid to the detection of narrow, soft seams that may be located at stratum boundaries.

For drilled shaft foundations, it is especially critical that the groundwater regime is well defined at each foundation location. Piezometer data adequate to define the limits and piezometric head in all unconfined, confined, and locally perched groundwater zones should be obtained at each foundation location.

Application	Minimum Number of Investigation Points and Location of Investigation Points	Minimum Depth of Investigation
Shallow Foundations	For substructure (e.g., piers or abutments) widths less than or equal to 100 feet, a minimum of one investigation point per substructure. For substructure widths greater than 100 feet, a minimum of two investigation points per substructure. Additional investigation points should be provided if erratic subsurface conditions are encountered. For cut-and-cover tunnels, culverts pipe arches, etc., spacing of investigation points shall be consistent for that required for retaining walls (see **WSDOT GDM Chapter 15**), with a minimum of two investigation points spaced adequately to develop a subsurface profile for the entire structure.	Depth of investigation should be: (1) Great enough to fully penetrate unsuitable foundation soils (e.g., peat, organic silt, soft fine grained soils) into competent material of suitable bearing capacity (e.g. stiff to hard cohesive soil, compact to dense cohesionless soil or bedrock) (2) At least to a depth where stress increase due to estimated foundation load is less than 10% of the existing effective overburden stress at that depth and; (3) If bedrock is encountered before the depth required by item (2) above is achieved, investigation depth should be great enough to penetrate a minimum of 10 feet into the bedrock, but rock investigation should be sufficient to characterize compressibility of infill material of near-horizontal to horizontal discontinuities.
Deep Foundations	For substructure (e.g., bridge piers or abutments) widths less than or equal to 100 feet, a minimum of one investigation point per substructure. For substructure widths greater than 100 feet, a minimum of two investigation points per substructure. Additional investigation points should be provided if erratic subsurface conditions are encountered. Due to large expense associated with construction of rock-socketed shafts, conditions should be confirmed at each shaft location.	In soil, depth of investigation should extend below the anticipated pile or shaft tip elevation a minimum of 20 feet, or a minimum of two times the maximum pile group dimension, whichever is deeper. All borings should extend through unsuitable strata such as unconsolidated fill, peat, highly organic materials, soft fine-grained soils, and loose coarse-grained soils to reach hard or dense materials, a minimum of 30 ft into soil with an average N-Value of 30 blows/ft or more. For piles bearing on rock, a minimum of 10 feet of rock core shall be obtained at each investigation point location to verify that the boring has not terminated on a boulder. For shafts supported on or extending into rock, a minimum of 10 feet of rock core, or a length of rock core equal to at least three times the shaft diameter for isolated shafts or two times the maximum shaft group dimension, whichever is greater, shall be extended below the anticipated shaft tip elevation to determine the physical characteristics of rock within the zone of foundation influence.

Table 8-2 Guidelines for Minimum Number of Investigation Points and Depth of Investigation (modified after Sabatini, et al., 2002).

8.3.2 Laboratory and Field Testing Requirements for Foundations

General requirements for laboratory and field testing, and their use in the determination of properties for design, are addressed in **WSDOT GDM Chapter 5**. In general, for foundation design, laboratory testing should be used to augment the data obtained from the field investigation program, to refine the soil and rock properties selected for design.

Foundation design will typically heavily rely upon the SPT and/or q_c results obtained during the field exploration through correlations to shear strength, compressibility, and the visual descriptions of the soil/rock encountered, especially in non-cohesive soils. The information needed for the assessment of ground water and the hydrogeologic properties needed for foundation design and constructability evaluation is typically obtained from the field exploration through field instrumentation (e.g., piezometers) and in-situ tests (e.g., slug tests, pump tests, etc.). Index tests such as soil gradation, Atterberg limits, water content, and organic content are used to confirm the visual field classification of the soils encountered, but may also be used directly to obtain input parameters for some aspects of foundation design (e.g., soil liquefaction, scour, degree of over-consolidation, and correlation to shear strength or compressibility of cohesive soils). Quantitative or performance laboratory tests conducted on undisturbed soil samples are used to assess shear strength or compressibility of finer grained soils, or to obtain seismic design input parameters such as shear modulus. Site performance data, if available, can also be used to assess design input parameters. Recommendations are provided in **WSDOT GDM Chapter 5** regarding how to make the final selection of design properties based on all of these sources of data.

8.4 Foundation Selection Considerations

Foundation selection considerations to be evaluated include:

- the ability of the foundation type to meet performance requirements (e.g., deformation, bearing resistance, uplift resistance, lateral resistance/deformation) for all limit states, given the soil or rock conditions encountered
- the constructability of the foundation type
- the impact of the foundation installation (in terms of time and space required) on traffic and right-of-way
- the environmental impact of the foundation construction
- the constraints that may impact the foundation installation (e.g., overhead clearance, access, and utilities)
- the impact of the foundation on the performance of adjacent foundations, structures, or utilities, considering both the design of the adjacent foundations, structures, or utilities, and the performance impact the installation of the new foundation will have on these adjacent facilities.
- the cost of the foundation, considering all of the issues listed above.

Spread footings are typically very cost effective, given the right set of conditions. Footings work best in hard or dense soils that have adequate bearing resistance and exhibit tolerable settlement under load. Footings can get rather large in medium dense or stiff soils to keep bearing stresses low enough to minimize settlement, or for structures with tall columns or which otherwise are loaded in a manner that results in large eccentricities at the footing level, or which result in the footing being subjected to uplift loads. Footings are not effective where soil liquefaction can occur at or below the footing level, unless the liquefiable soil is confined, not very thick, and well below the footing level. However, footings may be cost effective if inexpensive soil improvement techniques such as overexcavation, deep dynamic compaction, and stone columns, etc. are feasible. Other factors that affect the desirability of spread footings include the need for a cofferdam and seals when placed below the water table, the need for significant overexcavation of unsuitable soil, the need to place footings deep due to scour and possibly frost action, the need for significant shoring to protect adjacent existing facilities, and inadequate overall stability when placed on slopes that have marginally adequate stability. Footings may not be feasible where expansive or collapsible soils are present near the bearing elevation. Since deformation (service) often controls the feasibility of spread footings, footings may still be feasible and cost effective if the structure the footings support can be designed to tolerate the settlement (e.g., flat slab bridges, bridges with jackable abutments, etc.).

Deep foundations are the best choice when spread footings cannot be founded on competent soils or rock at a reasonable cost. At locations where soil conditions would normally permit the use of spread footings but the potential exists for scour, liquefaction or lateral spreading, deep foundations bearing on suitable materials below such susceptible soils should be used as a protection against these problems. Deep foundations should also be used where an unacceptable amount of spread footing settlement may occur. Deep foundations should be used where right-of-way, space limitations, or other constraints as discussed above would not allow the use of spread footings.

Two general types of deep foundations are typically considered: pile foundations, and drilled shaft foundations. Shaft foundations are most advantageous where very dense intermediate strata must be penetrated to obtain the desired bearing, uplift, or lateral resistance, or where obstructions such as boulders or logs must be penetrated. Shafts may also become cost effective where a single shaft per column can be used in lieu of a pile group with a pile cap, especially when a cofferdam or shoring is required to construct the pile cap. However, shafts may not be desirable where contaminated soils are present, since contaminated soil would be removed, requiring special handling and disposal. Shafts should be used in lieu of piles where deep foundations are needed and pile driving vibrations could cause damage to existing adjacent facilities. Piles may be more cost effective than shafts where pile cap construction is relatively easy, where the depth to the foundation layer is large (e.g., more than 100 ft), or where the pier loads are such that multiple shafts per column, requiring a shaft cap, are needed. The tendency of the upper loose soils to flow, requiring permanent shaft casing, may also be a consideration that could make pile foundations more cost effective. Artesian pressure in the bearing layer could preclude the use of drilled shafts due to the difficulty in keeping enough head inside the shaft during excavation to prevent heave or caving under slurry.

For situations where existing structures must be retrofitted to improve foundation resistance or where limited headroom is available, micro-piles may be the best alternative, and should be considered.

Augercast piles can be very cost effective in certain situations. However, their ability to resist lateral loads is minimal, making them undesirable to support structures where significant lateral loads must be transferred to the foundations. Furthermore, quality assurance of augercast pile integrity and capacity needs further development. Therefore, it is WSDOT policy not to use augercast piles for bridge foundations.

8.5 Overview of LRFD for Foundations

The basic equation for load and resistance factor design (LRFD) states that the loads multiplied by factors to account for uncertainty, ductility, importance, and redundancy must be less than or equal to the available resistance multiplied by factors to account for variability and uncertainty in the resistance per the AASHTO LRFD Bridge Design Specifications. The basic equation, therefore, is as follows:

$$\Sigma \eta_i \gamma_i Q_i \leq \varphi R_n \qquad (8\text{-}1)$$

η_i = Factor for ductility, redundancy, and importance of structure
γ_i = Load factor applicable to the i'th load Q_i
Q_i = Load
φ = Resistance factor
R_n = Nominal (predicted) resistance

For typical WSDOT practice, η_i should be set equal to 1.0 for use of both minimum and maximum load factors. Foundations shall be proportioned so that the factored resistance is not less than the factored loads.

Figure 8-2 below should be utilized to provide a common basis of understanding for loading locations and directions for substructure design. This figure also indicates the geometric data required for abutment and substructure design. Note that for shaft and some pile foundation designs, the shaft or pile may form the column as well as the foundation element, thereby eliminating the footing element shown in the figure.

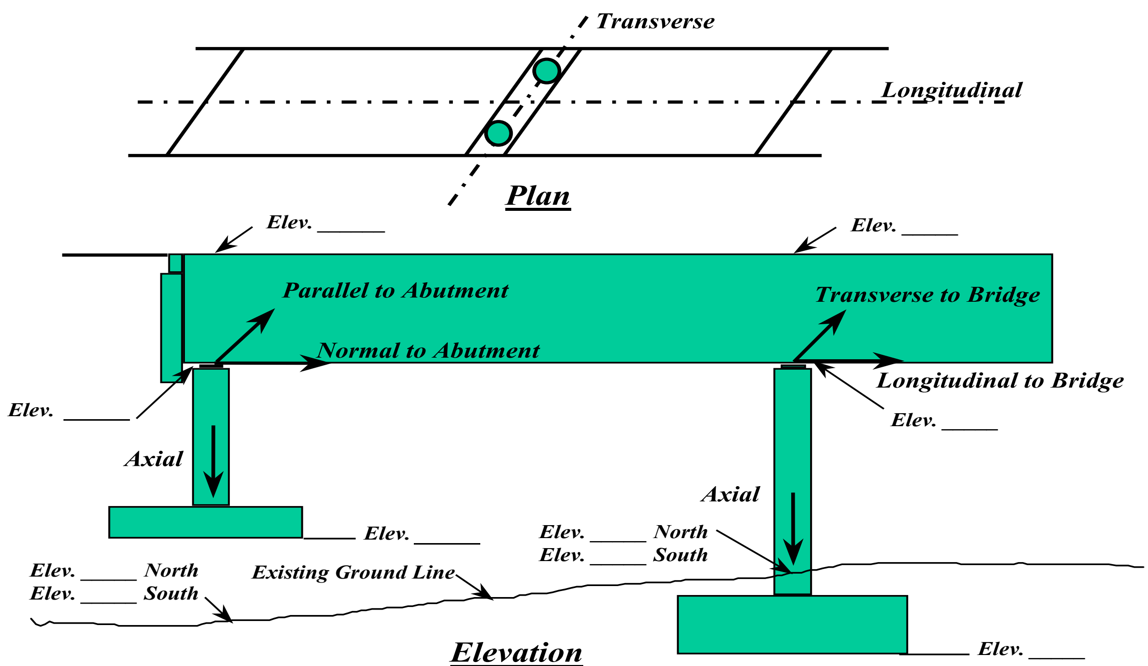


Figure 8-2 Template for foundation site data and loading direction definitions.

8.6 LRFD Loads, Load Groups and Limit States to be Considered

The specific loads and load factors to be used for foundation design are as found in AASHTO LRFD Bridge Design Specifications and the WSDOT LRFD Bridge Design Manual (BDM).

8.6.1 Foundation Analysis to Establish Load Distribution for Structure

Once the applicable loads and load groups for design have been established for each limit state, the loads shall be distributed to the various parts of the structure in accordance with Sections 3 and 4 of the AASHTO LRFD Bridge Design Specifications. The distribution of these loads shall consider the deformation characteristics of the soil/rock, foundation, and superstructure. The following process is used to accomplish the load distribution (see WSDOT LRFD BDM Section 7.2 for more detailed procedures):

1. Establish stiffness values for the structure and the soil surrounding the foundations and behind the abutments.
2. For service and strength limit state calculations, use P-y curves for deep foundations, or use strain wedge theory, especially in the case of short or intermediate length shafts (see **WSDOT GDM Section 8.13.4.7**), to establish soil/rock stiffness values (i.e., springs) necessary for structural design. The bearing resistance at the specified settlement determined for the service limit state, but excluding consolidation settlement, should be used to establish soil stiffness values for spread footings for service and strength limit state calculations. For strength limit state calculations for deep foundations where the lateral load is potentially repetitive in nature (e.g., wind, water, braking forces, etc.), use soil stiffness values derived from P-y curves using non-degraded soil strength and stiffness parameters. The geotechnical designer provides the soil/rock input parameters to the structural designer to develop these springs and to determine the load distribution using the analysis procedures as specified in WSDOT LRFD BDM Section 7.2 and Section 4 of the AASHTO LRFD Bridge Design Specifications, applying unfactored loads, to get the load distribution. Two unfactored load distributions for service and strength limit state calculations are developed: one using undegraded stiffness parameters (i.e., maximum stiffness values) to determine the maximum shear and moment in the structure, and another distribution using soil strength and stiffness parameters that have been degraded over time due to repetitive loading to determine the maximum deflections and associated loads that result.
3. For extreme event limit state (seismic) deep foundation calculations, use soil strength and stiffness values before any liquefaction or other time dependent degradation occurs to develop lateral soil stiffness values and determine the unfactored load distribution to the foundation and structure elements as described in Step 2, including the full seismic loading. This analysis using maximum stiffness values for the soil/rock is used by the structural designer to determine the maximum shear and moment in the structure. The structural designer then completes another unfactored analysis using soil parameters degraded by liquefaction effects to get another load distribution, again using the full seismic loading, to determine the maximum deflections and associated loads that result. For footing foundations, a similar process is followed, except the vertical soil springs are bracketed to evaluate both a soft response and a stiff response.
4. Once the load distributions have been determined, the loads are factored to analyze the various components of the foundations and structure for each limit state. The structural and geotechnical resistance are factored as appropriate, but in all cases, the lateral soil resistance for deep foundations remain unfactored (i.e., a resistance factor of 1.0).

Throughout all of the analysis procedures discussed above to develop load distributions, the soil parameters and stiffness values are unfactored. The geotechnical designer must develop a best estimate for these parameters during the modeling. Use of intentionally conservative values could result in unconservative estimates of structure loads, shears, and moments or inaccurate estimates of deflections.

See **WSDOT GDM Section 8.11.3.2** for the development of elastic settlement/bearing resistance of footings for static analyses and **WSDOT GDM Chapter 6** for soil/rock stiffness determination for spread footings subjected to seismic loads. See **WSDOT GDM Sections 8.12.2.5 and 8.13.4.7** for the development of lateral soil stiffness values for deep foundations.

8.6.2 Downdrag Loads

Regarding downdrag loads, possible development of downdrag on piles, shafts, or other deep foundations shall be evaluated where:

- Sites are underlain by compressible material such as clays, silts or organic soils,
- Fill will be or has recently been placed adjacent to the piles or shafts, such as is frequently the case for bridge approach fills,
- The groundwater is substantially lowered, or
- Liquefaction of loose sandy soil can occur.

When the potential exists for downdrag to act on a deep foundation, due to downward movement of the soil relative to the foundation, and the potential for downdrag is not eliminated by preloading the soil to reduce downward movements or other mitigating measure, the foundation shall be designed to resist the induced downdrag. This force effect is also termed negative skin friction.

The following load factors (γ_p) for downdrag (DD) shall be used at the strength limit state:

Type of Load, Foundation Type, and Method Used to Calculate Downdrag Load Factor		Maximum	Minimum
DD: Downdrag	Piles, α-Tomlinson Method	1.4	--
	Piles, λ-Method	1.05	--
	Drilled shafts, O'Neill and Reese (1999) Method	1.25	--

Table 8-3 Strength limit state load factors for downdrag, γ_p.

Regarding the load factors for downdrag in **Table 8-3**, use the maximum load factor when investigating maximum downward pile loads. The minimum load factor shall only be used when investigating possible uplift loads.

For some downdrag estimation methods, the magnitude of the load factor is dependent on the magnitude of the downdrag load relative to the dead load. The downdrag load factors were developed considering that downdrag loads equal to or greater than the magnitude of the dead load become somewhat impractical for design. See **Allen (2005)** for additional background and guidance on the effect of downdrag load magnitude.

For Extreme Event I limit state, downdrag induced by liquefaction settlement shall be applied to the foundation in combination with the other loads included within that load group. Liquefaction-induced downdrag shall not be combined with downdrag induced by consolidation settlements. For downdrag load applied to deep foundation groups, group effects shall be taken into account.

Downdrag can be caused by soil settlement due to loads applied after the foundation is constructed, such as an approach embankment as shown in **Figure 8-3**. Consolidation can also occur due to recent lowering of the ground water level as shown in **Figure 8-4**. Where piles are driven to end bearing on a dense stratum or rock and the design of the pile is structurally controlled, downdrag shall be evaluated at the strength and extreme limit states. For deep foundation elements that can experience settlement at the foundation tip, downdrag shall be evaluated at the service, strength and extreme limit states. In the case of friction elements with limited tip resistance, the downdrag load can exceed the geotechnical resistance of the foundation and cause the foundation to move downward. This design situation is not desirable and the preferred practice is to mitigate the settlement, and therefore downdrag potential. Consideration shall be given to eliminating the potential for static downdrag loads using embankment preloading plus surcharge loads, vertical drainage, and settlement monitoring measurements. The procedure for designing a preload is presented in **Cheney and Chassie (2000).**

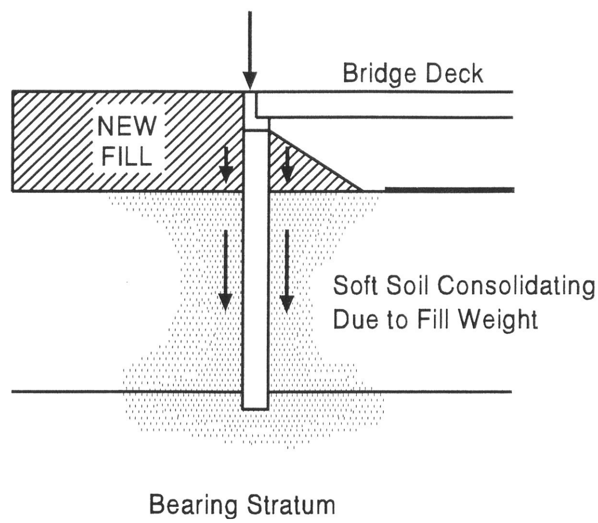


Figure 8-3 Common downdrag situation due to fill weight (Hannigan, et al. 1997).

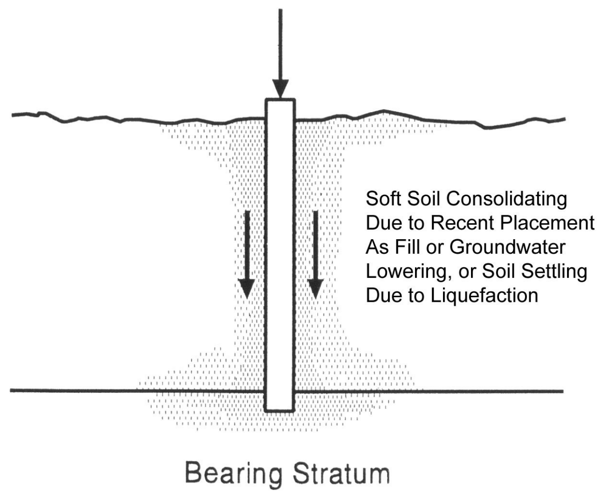


Figure 8-4 Common downdrag situation due to causes other than recent fill placement (Hannigan, et al. 1997).

Post-liquefaction settlement can also cause downdrag. Methods for mitigating liquefaction-induced downdrag are presented in **Kavazanjian, et al. (1997)**. For WSDOT projects, liquefaction mitigation should not rely on drainage techniques.

The application of downdrag to pile or shaft groups can be complex. If the pile or shaft cap is near or below the fill material causing consolidation settlement of the underlying soft soil, the cap will prevent transfer of stresses adequate to produce settlement of the soil inside the pile or shaft group. The downdrag applied in this case is the frictional force around the exterior of the pile or shaft group and along the sides of the pile or shaft cap (if any). If the cap is located well up in the fill causing consolidation stresses or if the piles or shafts are used as individual columns to support the structure above ground, the downdrag on each individual pile or shaft will control the magnitude of the load. If group effects are likely, the downdrag calculated using the group perimeter shear force should be determined in addition to the sum of the downdrag forces for each individual pile or shaft. The greater of the two calculations should be used for design.

The skin friction used to estimate downdrag due to liquefaction settlement should be conservatively assumed to be equal to the residual soil strength in the liquefiable zone, and non-liquefied skin friction in non-liquefiable layers above the zone of liquefaction.

If transient loads act to reduce the magnitude of downdrag loads and this reduction is considered in the design of the pile or shaft, the reduction shall not exceed that portion of transient load equal to the downdrag force effect. Transient loads can act to reduce the downdrag because they cause a downward movement of the pile resulting in a temporary reduction or elimination of the downdrag load. It is conservative to include the transient loads together with downdrag.

Force effects due to downdrag on deep foundations shall be computed as follows:

Step 1 – Establish soil profile and soil properties for computing settlement using the procedures in **WSDOT GDM Chapter 5** and in **Section 8.3**.

Step 2 – Perform settlement computations for the soil layers along the length of the deep foundation using the procedures in **WSDOT GDM Chapter 9** and **Section 8.11.3.2**. If the settlement is due to liquefaction, the **Tokimatsu and Seed (1987)** or **the Ishihara and Yoshimine (1992)** procedures should be used to estimate settlement.

Step 3 – Determine the length of deep foundation that will be subject to downdrag. If the settlement in the soil layer is 0.4 inches or greater, downdrag can be assumed to fully develop.

Step 4 – Determine the magnitude of the downdrag, DD, by computing the negative skin friction using the static analysis procedures in **WSDOT GDM Section 8.12.4.7.5** for piles and **WSDOT GDM Section 8.13.4.4** for shafts. Sum the negative skin friction for all layers contributing to downdrag from the lowest layer to the bottom of the pile cap or ground surface.

The methods used to estimate downdrag are the same as those used to estimate skin friction. The distinction between the two is that downdrag acts downward on the sides of the piles or shafts and loads the foundation, whereas skin friction acts upward on the sides of piles or shafts and, thus, supports the foundation loads.

Downdrag for piles can be estimated using the α or λ methods for cohesive soils. An alternative approach would be to use the β method where the long-term conditions after consolidation should be considered. Cohesionless soil layers overlying the consolidating layers will also contribute to downdrag and the negative skin friction in these layers should be estimated using an effective stress method.

Downdrag loads for shafts may be estimated using the α-method for cohesive soils and the β-method for granular soils, as specified in **WSDOT GDM Section 8.13.4.4.1(a)**, for calculating negative shaft resistance. As with positive shaft resistance, the top 5.0 FT and a bottom length taken as one shaft diameter do not contribute to downdrag loads. When using the α-method, an allowance should be made for a possible increase in the undrained shear strength as consolidation occurs. If the downdrag is due to liquefaction, the β-method as described in **WSDOT GDM Section 8.13.4.4.2(a)** should be used to estimate the downdrag load for drilled shafts using non-liquefied soil properties.

The neutral plane method may also be used to determine downdrag. The neutral plane method is described and discussed in NCHRP 393 (**Briaud and Tucker, 1993**).

8.6.3 Uplift Loads due to Expansive Soils

In general, uplift loads on foundations due to expansive soils shall be avoided through removal of the expansive soil. If removal is not possible, deep foundations such as driven piles or shafts shall be placed into stable soil. Spread footings shall not be used in this situation.

Deep foundations penetrating expansive soil shall extend to a depth into moisture-stable soils sufficient to provide adequate anchorage to resist uplift. Sufficient clearance should be provided between the ground surface and underside of caps or beams connecting piles or shafts to preclude the application of uplift loads at the pile/cap connection due to swelling ground conditions.

Evaluation of potential uplift loads on piles extending through expansive soils requires evaluation of the swell potential of the soil and the extent of the soil strata that may affect the pile. One reasonably reliable method for identifying swell potential is presented in **WSDOT GDM Chapter 5.** Alternatively, ASTM D4829 may be used to evaluate swell potential. The thickness of the potentially expansive stratum must be identified by:

- Examination of soil samples from borings for the presence of jointing, slickensiding, or a blocky structure and for changes in color, and
- Laboratory testing for determination of soil moisture content profiles.

8.6.4 Soil Loads on Buried Structures

For tunnels, culverts and pipe arches, the soil loads to be used for design shall be as specified in Sections 3 and 12 of the AASHTO LRFD Bridge Design Specifications.

8.6.5 Service Limit States

Foundation design at the service limit state shall include:

- Settlements
- Horizontal movements
- Overall stability, and
- Scour at the design flood

Consideration of foundation movements shall be based upon structure tolerance to total and differential movements, rideability and economy. Foundation movements shall include all movement from settlement, horizontal movement, and rotation.

In bridges where the superstructure and substructure are not integrated, settlement corrections can be made by jacking and shimming bearings. Article 2.5.2.3 of AASHTO LRFD Bridge Design Specifications requires jacking provisions for these bridges. The cost of limiting foundation movements should be compared with the cost of designing the superstructure so that it can tolerate larger movements or of correcting the consequences of movements through maintenance to determine minimum lifetime cost. WSDOT may establish criteria that are more stringent.

The design flood for scour is defined in Article 2.6.4.4.2 and is specified in Article 3.7.5 of the AASHTO LRFD Bridge Design Specifications as applicable at the service limit state.

8.6.5.1 Tolerable Movements

Foundation settlement, horizontal movement, and rotation of foundations shall be investigated using all applicable loads in the Service I Load Combination specified in Table 3.4.1-1 of the AASHTO LRFD Bridge Design Specifications. Transient loads may be omitted from settlement analyses for foundations bearing on or in cohesive soil deposits that are subject to time-dependant consolidation settlements.

Foundation movement criteria shall be consistent with the function and type of structure, anticipated service life, and consequences of unacceptable movements on structure performance. Foundation movement shall include vertical, horizontal and rotational movements. The tolerable movement criteria shall be established by either empirical procedures or structural analyses or by consideration of both.

Experience has shown that bridges can and often do accommodate more movement and/or rotation than traditionally allowed or anticipated in design. Creep, relaxation, and redistribution of force effects accommodate these movements. Some studies have been made to synthesize apparent response. These studies indicate that angular distortions between adjacent foundations greater than 0.008 (RAD) in simple spans and 0.004 (RAD) in continuous spans should not be permitted in settlement criteria (**Moulton et al. 1985; DiMillio, 1982; Barker et al. 1991**). Other angular distortion limits may be appropriate after consideration of:

- Cost of mitigation through larger foundations, realignment or surcharge,
- Rideability,
- Aesthetics, and,
- Safety.

In addition to the requirements for serviceability provided above, the following criteria (**Tables 8-4, 8-5, and 8-6**) shall be used to establish acceptable settlement criteria:

Total Settlement at Pier or Abutment	Differential Settlement Over 100 ft within Pier or Abutment, and Differential Settlement Between Piers	Action
$\Delta H \leq 1$ in	$\Delta H_{100} \leq 0.75$ in	Design and Construct
1 in $< \Delta H \leq 4$ in	0.75 in $< \Delta H_{100} \leq 3$ in	Ensure structure can tolerate settlement
$\Delta H > 4$ in	$\Delta H_{100} > 3$ in	Obtain Approval[1] prior to proceeding with design and Construction

[1]Approval of WSDOT State Geotechnical Engineer and WSDOT Bridge Design Engineer required.

Table 8-4 Settlement criteria for bridges.

Total Settlement	Differential Settlement Over 100 ft	Action
$\Delta H \leq 1$ in	$\Delta H_{100} \leq 0.75$ in	Design and Construct
1 in $< \Delta H \leq 2.5$ in	0.75 in $< \Delta H_{100} \leq 2$ in	Ensure structure can tolerate settlement
$\Delta H > 2.5$ in	$\Delta H_{100} > 2$ in	Obtain Approval[1] prior to proceeding with design and Construction

[1]Approval of WSDOT State Geotechnical Engineer and WSDOT Bridge Design Engineer required.

Table 8-5 Settlement criteria for cut and cover tunnels, concrete culverts (including box culverts), and concrete pipe arches.

Total Settlement	Differential Settlement Over 100 ft	Action
$\Delta H \leq 2$ in	$\Delta H_{100} \leq 1.5$ in	Design and Construct
2 in $< \Delta H \leq 6$ in	1.5 in $< \Delta H_{100} \leq 5$ in	Ensure structure can tolerate settlement
$\Delta H > 6$ in	$\Delta H_{100} > 5$ in	Obtain Approval[1] prior to proceeding with design and Construction

[1]Approval of WSDOT State Geotechnical Engineer and WSDOT Bridge Design Engineer required.

Table 8-6 Settlement criteria for flexible culverts.

Rotation movements should be evaluated at the top of the substructure unit (in plan location) and at the deck elevation.

The horizontal displacement of pile and shaft foundations shall be estimated using procedures that consider soil-structure interaction (see **WSDOT GDM Section 8.12.2.5**). Horizontal movement criteria should be established at the top of the foundation based on the tolerance of the structure to lateral movement, with consideration of the column length and stiffness. Tolerance of the superstructure to lateral movement will depend on bridge seat widths, bearing type(s), structure type, and load distribution effects.

8.6.5.2 Overall Stability

The evaluation of overall stability of earth slopes with or without a foundation unit shall be investigated at the service limit state as specified in Article 11.6.3.4 of the AASHTO LRFD Bridge Design Specifications. Overall stability should be evaluated using limiting equilibrium methods such as modified Bishop, Janbu, Spencer, or other widely accepted slope stability analysis methods. Article 11.6.3.4 recommends that overall stability be evaluated at the Service I limit state (i.e., a load factor of 1.0) and a resistance factor, ϕ_{OS} of 0.65 for slopes which support a structural element. For resistance factors for overall stability of slopes that contain a retaining wall, see **WSDOT GDM Chapter 15**. Also see **WSDOT GDM Chapter 7** for additional information and requirements regarding slope stability analysis and acceptable safety factors and resistance factors.

Available slope stability programs produce a single factor of safety, FS. Overall slope stability shall be checked to insure that foundations designed for a maximum bearing stress equal to the specified service limit state bearing resistance will not cause the slope stability factor of safety to fall below 1.5. This practice will essentially produce the same result as specified in Article 11.6.3.4 of the AASHTO LRFD Bridge Design Specifications. The foundation loads should be as specified for the Service I limit state for this analysis. If the foundation is located on the slope such that the foundation load contributes to slope instability, the designer shall establish a maximum footing load that is acceptable for maintaining overall slope stability for Service, and Extreme Event limit states (see **Figure 8-5** for example). If the foundation is located on the slope such that the foundation load increases slope stability, overall stability of the slope shall evaluated ignoring the effect of the footing on slope stability, or the foundation load shall be included in the slope stability analysis and the foundation designed to resist the lateral loads imposed by the slope.

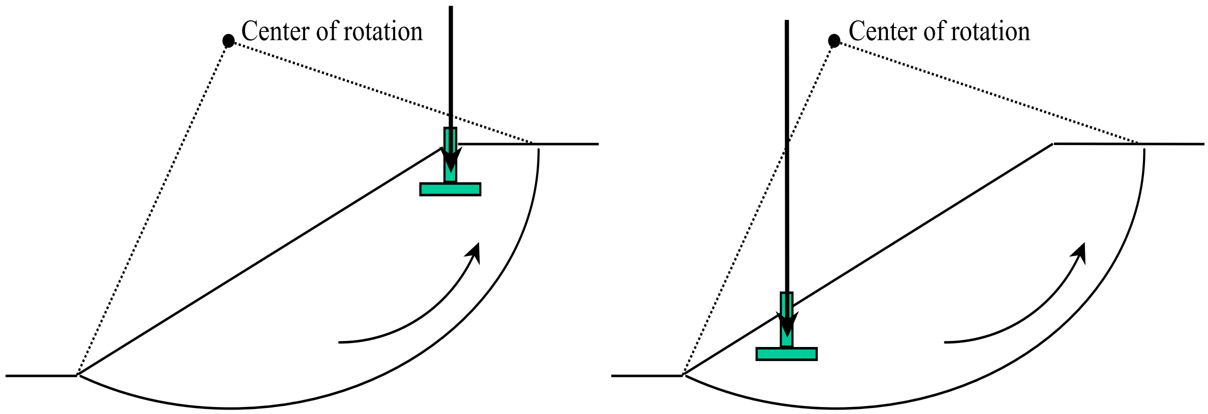


Figure 8-5 Example where footing contributes to instability of slope (left figure) vs. example where footing contributes to stability of slope (right figure).

8.6.5.3 Abutment Transitions

Vertical and horizontal movements caused by embankment loads behind bridge abutments shall be investigated. Settlement of foundation soils induced by embankment loads can result in excessive movements of substructure elements. Both short and long term settlement potential should be considered.

Settlement of improperly placed or compacted backfill behind abutments can cause poor rideability and a possibly dangerous bump at the end of the bridge. Guidance for proper detailing and material requirements for abutment backfill is provided in **Cheney and Chassie (2000)** and should be followed.

Lateral earth pressure behind and/or lateral squeeze below abutments can also contribute to lateral movement of abutments and should be investigated, if applicable.

In addition to the considerations for addressing the transition between the bridge and the abutment fill provided above, an approach slab shall be provided at the end of each bridge for WSDOT projects, and shall be the same width as the bridge deck. However, the slab may be deleted under certain conditions as described herein. If approach slabs are to be deleted, a geotechnical and structural evaluation is required. The final decision on whether or not to delete the approach slabs shall be made by the WSDOT Region Project Development Engineer with consideration to the geotechnical and structural evaluation. The geotechnical and structural evaluation shall consider, as a minimum, the criteria described below.

1. Approach slabs may be deleted for geotechnical reasons if the following geotechnical considerations are met:
   - If settlements are excessive, resulting in the angular distortion of the slab to be great enough to become a safety problem for motorists, with excessive defined as a differential settlement between the bridge and the approach fill of 8 inches or more, or,
   - If creep settlement of the approach fill will be less than 0.5 inch, and the amount of new fill placed at the approach is less than 20 ft, or
   - If approach fill heights are less than 8 ft, or
   - If more than 2 inches of differential settlement could occur between the centerline and shoulder

2. Other issues such as design speed, average daily traffic (ADT) or accommodation of certain bridge structure details may supersede the geotechnical reasons for deleting the approach slabs. Approach slabs shall be used for all WSDOT bridges with stub abutments to accommodate bridge expansion and contraction. Approach slabs are not required for accommodating expansion and contraction of the bridge for "L" abutments. For bridge widenings, approach slabs shall be provided for the widening if the existing bridge has an approach slab. If the existing bridge does not have an approach slab, and it is not intended to install an approach slab for the full existing plus widened bridge width, an approach slab shall not be provided for the bridge widening.

8.6.6 Strength Limit States

Design of foundations at strength limit states shall include evaluation of the nominal geotechnical and structural resistances of the foundation elements. Design at strength limit states shall not consider the deformations required to mobilize the nominal resistance, unless a definition of failure based on deflection is specified.

The geotechnical design of all foundations at the strength limit state shall consider:

- Loss of lateral support; and
- Scour at the design flood event.

For the purpose of design at strength limit states, the nominal resistance is considered synonymous with the ultimate capacity of an element as previously defined under allowable stress design (i.e., **AASHTO 2002**). For design of foundations such as piles or drilled shafts that may be based directly on static load tests, or correlation to static load tests, the definition of failure may include a deflection-limited criteria.

The design event for scour is defined in Section 2 of the AASHTO LRFD Bridge Design Specifications and is specified in AASHTO LRFD Article 3.7.5 as applicable at the strength limit state.

8.6.6.1 Spread Footings

The design of spread footings at the strength limit state shall also include:

- Nominal bearing resistance;
- Overturning or excessive loss of contact;
- Sliding at the base of footing; and
- Constructability.

The designer should assess whether special construction methods are required to bear a spread footing at the design depth. Consideration should be given to the potential need for shoring, cofferdams, seals, and/or dewatering. Basal stability of excavations should be evaluated, particularly if dewatering or cofferdams are required.

The presence of expansive/collapsible soils in the vicinity of the footing should be identified. If present, the design of the footing should be modified to accommodate the potential impact to the performance of the structure or the expansive/collapsible soils should be mitigated.

8.6.6.2 Driven Piles

The design of pile foundations at the strength limit state shall also include:

- Axial compression resistance for single piles
- Pile group compression resistance
- Uplift resistance for single piles
- Uplift resistance for pile groups
- Single pile and pile group lateral resistance
- Pile punching failure into a weaker stratum below the bearing stratum, and
- Constructability, including pile drivability.

For pile foundations, as part of the evaluation for the strength limit states identified herein, the effects of downdrag, soil setup or relaxation, and buoyancy due to groundwater should be evaluated.

8.6.6.3 Drilled Shafts

The design of drilled shaft foundations at the strength limit state shall also include:

- Axial compression resistance for single drilled shafts
- Shaft group compression resistance
- Uplift resistance for single shafts
- Uplift resistance for shaft groups
- Single shaft and shaft group lateral resistance
- Shaft punching failure into a weaker stratum below the bearing stratum, and
- Constructability, including method(s) of shaft construction.

The design of drilled shafts for each of these limit states should include the effects of the method of construction, including construction sequencing, whether the shaft will be excavated in the dry or if wet methods must be used, as well as the need for temporary or permanent casing to control caving ground conditions, and the effects of downdrag. The design assumptions regarding construction methods must carry through to the contract documents to provide assurance that the geotechnical and structural resistance used for design will be provided by the constructed product.

8.6.7 Extreme Event Limit States

Foundations shall be designed for extreme events as applicable. Extreme events include the check flood for scour, vessel and vehicle collision, seismic loading, and other site-specific situations that the Engineer determines should be included. Refer to Section 10, Appendix A in the AASHTO LRFD Bridge Design Specifications for guidance regarding seismic analysis and earthquake design of foundations.

8.7 Resistance Factors for Foundation Design – Design Parameters

The load and resistance factors provided herein result from a combination of design model uncertainty, soil/rock property uncertainty, and unknown uncertainty assumed by the previous allowable stress design and load factor design approach included in previous AASHTO design specifications. Therefore, the load and resistance factors account for soil/rock property uncertainty in addition to other uncertainties.

It should be assumed that the characteristic soil/rock properties to be used in conjunction with the load and resistance factors provided herein that have been calibrated using reliability theory (see **Allen, 2005**) are average values obtained from laboratory test results or from correlated field in-situ test results. It should be noted that use of lower bound soil/rock properties could result in overly conservative foundation designs in such cases. However, depending on the availability of soil or rock property data and the variability of the geologic strata under consideration, it may not be possible to reliably estimate the average value of the properties needed for design. In such cases, the geotechnical designer may have no choice but to use a more conservative selection of design parameters to mitigate the additional risks created by potential variability or the paucity of relevant data. Regarding the extent of subsurface characterization and the number of soil/rock property tests required to justify use of the load and resistance factors provided herein, see **WSDOT GDM Chapter 5**. For those load and resistance factors determined primarily from calibration by fitting to allowable stress design, this property selection issue is not relevant, and property selection should be based on past practice. For information regarding the deprivation of load and resistance factors for foundations, **(see Allen, 2005)**

8.8 Resistance Factors for Foundation Design – Service Limit States

Resistance factors for the service limit states shall be taken as 1.0, except as provided for overall stability in **WSDOT GDM Section 8.6.5.2**.

A resistance factor of 1.0 shall be used to assess the ability of the foundation to meet the specified deflection criteria after scour due to the design flood.

8.9 Resistance Factors for Foundation Design – Strength Limit States

Resistance factors for different types of foundation systems at the strength limit state shall be taken as specified in **Tables 8-7, 8-8, and 8-9**, unless regionally specific values are available. Regionally specific values should be determined based on substantial statistical data combined with calibration or substantial successful experience to justify higher values. Smaller resistance factors should be used if site or material variability is anticipated to be unusually high or if design assumptions are required that increase design uncertainty that have not been mitigated through conservative selection of design parameters.

METHOD/SOIL/CONDITION			RESISTANCE FACTOR
Bearing Resistance	φ_b	Theoretical method (**Munfakh, et al., 2001**), in clay	0.50
		Theoretical method (**Munfakh, et al., 2001**), in sand, using CPT	0.50
		Theoretical method (**Munfakh, et al., 2001**), in sand, using SPT	0.45
		Semi-empirical methods (**Meyerhof, 1956**), all soils	0.45
		Footings on rock	0.45
		Plate Load Test	0.55
Sliding	φ_T	Precast concrete placed on sand	0.90
		Cast-in-Place Concrete on sand	0.80
		Cast-in-Place or precast Concrete on Clay	0.85
		Soil on soil	0.90
	φ_{ep}	Passive earth pressure component of sliding resistance	0.50

Table 8-7 Resistance factors for geotechnical resistance of shallow foundations at the strength limit state.

CONDITION/RESISTANCE DETERMINATION METHOD		RESISTANCE FACTOR
Nominal Resistance of Single Pile in Axial Compression – Dynamic Analysis and Static Load Test Methods (φ_{dyn})	Driving criteria established by static load test(s); quality control by dynamic testing and/or calibrated wave equation, or minimum driving resistance combined with minimum delivered hammer energy from the load test(s). For the last case, the hammer used for the test pile(s) shall be used for the production piles.	Values in **Table 8-9**
	Driving criteria established by dynamic test with signal matching at beginning of redrive conditions only of at least one production pile per pier, but no less than the number of tests per site provided in **Table 8-10**. Quality control of remaining piles by calibrated wave equation and/or dynamic testing.	0.65
	Wave equation analysis without pile dynamic measurements	0.40
	WSDOT Driving formula (end of drive conditions only)	0.55
Nominal Resistance of Single Pile in Axial Compression – Static Analysis Methods (φ_{stat})	Skin Friction and End Bearing: Clay and Mixed Soils α-method (**Tomlinson, 1987; Skempton, 1951**) β-method (**Esrig & Kirby, 1979; Skempton, 1951**) λ-method (**Vijayvergiya & Focht, 1972; Skempton, 1951**) Skin Friction and End Bearing: Sand Nordlund/Thurman Method (**Hannigan, et al., 1997**) SPT-method – (**Meyerhof, 1976**) CPT-method (**Nottingham and Schmertmann, 1975**) End bearing in rock (**Canadian Geotech. Society, 1985**)	 0.35 0.25 0.40 0.45 0.30 0.50 0.45
Block Failure (φ_{bl})	Clay	0.60
Uplift Resistance of Single Piles (φ_{up})	α-method β-method λ-method Nordlund method SPT-method CPT-method Load Test	0.25 0.20 0.30 0.35 0.25 0.40 0.60
Group Uplift Resistance (φ_{ug})	Sand & Clay	0.50
Horizontal Geotechnical Resistance of Single Pile or Pile Group	All soils and rock	1.0
Pile Drivability Analysis, φ_{da}	Steel Piles — See the provisions of Article 6.5.4.2 (AASHTO LRFD Spec's.) Concrete Piles — See the provisions of Article 5.5.4.2.1 (AASHTO LRFD Spec's) Timber Piles — See the provisions of Article 8.5.2.2 (AASHTO LRFD Spec's.) In all three AASHTO LRFD Bridge Design Specification articles identified above, use φ identified as "resistance during pile driving".	

Table 8-8 Resistance factors for driven piles.

Number of Static Load Tests per Site	Resistance Factor, φ		
	Site Variability*		
	Low*	Medium*	High*
1	0.80	0.70	0.55
2	0.90	0.75	0.65
3	0.90	0.85	0.75
≥ 4	0.90	0.90	0.80

*See Paikowsky, et al. (2004) and discussion herein for guidelines on how to assess site variability.

Table 8-9 Relationship between Number of Static Load Tests Conducted per Site and φ (after Paikowsky, et al., 2004)

Site Variability*	Low*	Medium*	High*
Number of Piles Located within Site	Number of Piles with Dynamic Tests and Signal Matching Analysis Required (BOR)		
≤ 15	3	4	6
16-25	3	5	8
26-50	4	6	9
51-100	4	7	10
101-500	4	7	12
> 500	4	7	12

*See Paikowsky, et al. (2004) and discussion herein for guidelines on how to assess site variability.

Table 8-10 Number of Dynamic Tests with Signal Matching Analysis per Site to Be Conducted During Production Pile Driving (after Paikowsky, et al., 2004)

METHOD/SOIL/CONDITION			RESISTANCE FACTOR
Nominal Axial Compressive Resistance of Single-Drilled Shafts, φ_{stat}	Side Resistance in Clay	α-method (***O'Neill and Reese 1999***)	0.45
	Tip Resistance in Clay	Total Stress (***O'Neill and Reese 1999***)	0.40
	Side Resistance in Sand	β-method (**O'Neill and Reese *1999***)	0.55
	Tip Resistance in Sand	**O'Neill and Reese (*1999*)**	0.50
	Side Resistance in IGM's	**O'Neill and Reese (*1999*)**	0.60
	Tip Resistance in IGM's	**O'Neill and Reese (*1999*)**	0.55
	Side Resistance in Rock	**Horvath and Kenney (*1979*)** **O'Neill and Reese (*1999*)**	0.55 0.55
	Side Resistance in Rock	**Carter and Kulhawy (1988)**	0.50
	Tip Resistance in Rock	**Canadian Geotechnical Society (*1985*)** **Pressuremeter Method (*Canadian Geotechnical Society 1985*)** **O'Neill and Reese (*1999*)**	0.50 0.50 0.50
Block Failure, φ_{bl}	Clay		0.55
Uplift Resistance of Single-Drilled Shafts, φ_{up}	Clay	α-method (***O'Neill and Reese 1999***)	0.35
	Sand	β-method (**O'Neill and Reese *1999***)	0.45
	Rock	**Horvath and Kenney (*1979*)** **Carter and Kulhawy (1988)**	0.40
Group Uplift Resistance, φ_{ug}	Sand & Clay		0.45
Horizontal Geotechnical Resistance of Single Shaft or Shaft Group	All materials		1.0
Static Load Test (compression), φ_{load}	All Materials		Values in **Table 8-9**, but no greater than 0.70
Static Load Test (uplift), φ_{upld}	All Materials		0.60

Table 8-11 Resistance factors for geotechnical resistance of drilled shafts.

The nominal foundation resistance after scour due to the design flood shall provide adequate foundation resistance using the resistance factors given in this section. Scour design for the 100-year flood must satisfy the requirement that the nominal foundation resistance after scour is greater than the factored load determined with the scoured soil removed. The resistance factors will be those used in the Strength Limit State, without scour.

Certain resistance factors in **Tables 8-7, 8-8 and 8-11** are presented as a function of soil type (e.g., sand or clay). Naturally occurring soils do not fall neatly into these two classifications. In general, the terminology "sand" and "cohesionless soil" should be connoted to mean drained conditions during loading, while "clay" or "cohesive soil" implies undrained conditions. For other or intermediate soil classifications, such as silts or gravels, the designer should choose, depending on the load case under consideration, whether the resistance provided by the soil will be a drained or undrained strength, and select the method of computing resistance and associated resistance factor accordingly.

In general, resistance factors for bridge and other structure design have been derived to achieve a reliability index, β, of 3.5 (i.e., an approximate probability of failure, P_f, of 1 in 5,000). However, past geotechnical design practice has resulted in an effective reliability index, β, of 3.0 (probability of failure of approximately 1 in 1,000) for foundations in general, and for highly redundant systems, such as pile groups, an approximate reliability index, β, of 2.3 (an approximate probability of failure of 1 in 100) (**Zhang, et al., 2001; Paikowsky, et al., 2004; Allen, 2005**). If the resistance factors provided in this article are adjusted to account for regional practices using statistical data and calibration, they should be developed using the β values provided above, with consideration given to the redundancy in the foundation system.

For bearing resistance, lateral resistance, and uplift calculations, the focus of the calculation is on the individual foundation element (e.g., a single pile or drilled shaft). Since these foundation elements are usually part of a foundation unit that contains multiple elements, failure of one of these foundation elements usually does not cause the entire foundation unit to reach failure (i.e., due to load sharing and overall redundancy). Therefore, the reliability of the foundation unit is usually more, and in many cases considerably more, than the reliability of the individual foundation element. Hence, a lower reliability can be successfully used for redundant foundations than is typically the case for the superstructure.

Note that not all of the resistance factors provided in this section have been derived using statistical data from which a specific β value can be estimated, since such data were not always available. In those cases, where data were not available, resistance factors were estimated through calibration by fitting to past allowable stress design safety factors (e.g., the **AASHTO Standard Specifications for Highway Bridges, 2002**).

Additional discussion regarding the basis for the resistance factors for each foundation type and limit state is provided in **WSDOT GDM Sections 8.9.1, 8.9.2, and 8.9.3**. Additional, more detailed information on the development of the resistance factors for foundations provided herein, and a comparison of those resistance factors to previous Allowable Stress Design practice (e.g., **AASHTO 2002**) is provided in **Allen (2005)**.

8.9.1 Resistance Factor Considerations for Spread Footings

The resistance factors in **Table 8-7** were developed using both reliability theory and calibration by fitting to Allowable Stress Design (ASD). In general, ASD safety factors for footing bearing capacity range from 2.5 to 3.0, corresponding to a resistance factor of approximately 0.55 to 0.45, respectively, and for sliding, an ASD safety factor of 1.5, corresponding to a resistance factor of approximately 0.9. Calibration by fitting to ASD controlled the selection of the resistance factor in cases where statistical data were limited in quality or quantity. The resistance factor for sliding of cast-in-place concrete on sand is slightly lower than the other sliding resistance factors based on reliability theory analysis (**Barker, et al., 1991**). The higher interface friction coefficient used for sliding of cast-in-place concrete on sand relative to that used for precast concrete on sand causes the cast-in-place concrete sliding analysis to be less conservative, resulting in the need for the lower resistance factor. A more detailed explanation of the development of the resistance factors provided in **Table 8-7** is provided in **Allen (2005)**.

The resistance factors for plate load tests and passive resistance were based on engineering judgment. Regarding plate load tests, extrapolation of the plate load test data to a full scale footing should be based on the design procedures provided in **WSDOT GDM Section 8.11** for settlement (service limit state) and bearing resistance (strength and extreme event limit state), with consideration to the effect of the stratification (i.e., layer thicknesses, depths, and properties). Plate load test results shall be applied only within a sub-area of the project site for which the subsurface conditions (i.e., stratification, geologic history, properties) are relatively uniform.

8.9.2 Resistance Factor Considerations for Driven Piles

Resistance factors shall be selected from **Table 8-8** based on the method used for determining the nominal axial pile resistance. If pile resistance is verified in the field using a dynamic method such as a driving formula, or dynamic measurements combined with signal matching, the resistance factor for the field verification method should be used to determine the number of piles of a given nominal resistance needed to resist the factored loads in the strength limit state. Where nominal pile axial resistance is determined during pile driving by dynamic analysis, dynamic formulae, or static load test, the uncertainty in the pile axial resistance is strictly due to the reliability of the resistance determination method used in the field during pile installation.

In most cases, the nominal bearing resistance of each pile is field verified using a dynamic method (e.g., see **WSDOT GDM Sections 8.12.4.7.1, 8.12.4.7.2, 8.12.4.7.3, or 8.12.4.7.4**). The actual penetration depth where the pile is stopped using the results of the dynamic analysis will likely not be the same as the estimated depth from the static analysis. Hence, the reliability of the pile bearing resistance is dependent on the reliability of the method used to verify the bearing resistance during pile installation (see **Allen, 2005**, for additional discussion on this issue). Once the number of piles with a given nominal resistance needed to resist the factored loads is determined, the estimated depth of pile penetration to obtain the desired resistance is determined using the resistance factor for the static analysis method, equating the factored static analysis resistance to the factored dynamic analysis resistance (see **WSDOT GDM Section 8.12.4.2**).

Dynamic methods may be unsuitable for field verification of nominal axial resistance if soft silts or clays where a large amount of setup is anticipated and it is not feasible to obtain dynamic measurement of pile restrikes over a sufficient length of time to assess soil setup. Dynamic methods may not be applicable for determination of axial resistance when driving piles to rock (see **WSDOT GDM Section 8.12.4.1**).

Regarding load tests, and dynamic tests with signal matching, the number of tests to be conducted to justify the resistance factors provided in **tables 8-8, 8-9, and 8-10** should be based on the variability in the properties and geologic stratification of the site to which the test results are to be applied. A site shall be defined as a project site, or a portion of it, where the subsurface conditions can be characterized as geologically similar in terms of subsurface stratification (i.e., sequence, thickness, and geologic history of strata), the engineering properties of the strata, and groundwater conditions. Note that a site as defined herein may be only a portion of the area in which the structure (or structures) is located. For sites where conditions are highly variable, a site could even be limited to a single pier.

One or more static load tests should be performed per site to justify using the resistance factors in **Table 8-9** for piles installed within the site. **Tables 8-9 and 8-10** identify resistance factors to be used and numbers of tests needed depending on whether the site variability is classified as low, medium, or high. Site variability may be determined based on judgment, or based on the following suggested criteria (**Paikowsky, et al., 2004**):

Step 1: For each identified significant stratum at each boring location, determine the average property value (e.g., SPT value, q_c value, etc.) within the stratum for each boring.

Step 2: Determine the mean and coefficient of variation of the average values for each stratum determined in Step 1.

Step 3: Categorize the site variability as low if the COV is less than 25%, medium of the COV is 25% or more, but less than 40%, and high if the COV is 40% or more.

See **Paikowsky, et al. (2004**) for additional discussion regarding these site variability criteria.

The dynamic testing with signal matching (see **WSDOT GDM Section 8.12.4.7.2**) should be evenly distributed within a pier and across the entire structure in order to justify the use of the specified resistance factors. However, within a particular footing a considerable increase in safety is realized where the most heavily loaded piles are tested. See **Paikowsky, et al. (2004)** for additional guidelines regarding the number of production piles that should be tested using dynamic measurements in consideration of the site variability to justify the use of the specified resistance factors.

To be consistent with the calibration conducted to determine the resistance factors in **Tables 8-8, 8-9, and 8-10**, the signal matching analysis **(Rausche, et al., 1972)** of the dynamic test data should be conducted as described in **Hannigan, et al. (1997)**.

The dynamic pile formula identified in **Table 8-8** require the pile hammer energy as an input parameter. The developed hammer energy should be used for this purpose, defined as the product of actual stroke developed during the driving of the pile (or equivalent stroke as determined from the bounce chamber pressure for double acting hammers) and the hammer ram weight.

For all axial resistance calculation methods, the resistance factors were in general developed from load test results obtained on piles with diameters of 24 inches or less. Very little data were available for larger diameter piles. Therefore, these resistance factors should be used with caution for design of significantly larger diameter piles.

Paikowsky, et al. (2004) indicate that the resistance factors for static pile resistance analysis methods can vary significantly for different pile types. The resistance factors presented are average values for the method. See **Paikowsky, et al. (2004)** and **Allen (2005)** for additional information regarding this issue.

The resistance factor for the Nordlund/Thurman method was derived primarily using the **Peck, et al. (1974)** correlation between SPT $N1_{60}$ and the soil friction angle, using a maximum design soil friction angle of 36^{o}, assuming the contributing zone for the end bearing resistance is from the tip to 2 pile diameters below the tip.

For the clay static pile analysis methods, if the soil cohesion was not measured in the laboratory, the correlation between SPT N and S_u by **Hara, et al. (1974)** was used for the calibration. Use of other methods to estimate S_u may require the development of resistance factors based on those methods.

For the statistical calibrations using reliability theory, a target reliability index, β, of 2.3 (an approximate probability of failure of 1 in 100) was used. The selection of this target reliability assumes a significant amount of redundancy in the foundation system is present, which is typical for pile groups containing at least 5 piles in the group (**Paikowsky, et al., 2004**). For smaller groups and single piles, less redundancy will be present. The issue of redundancy, or the lack of it, is addressed in Article 1.3.4 of the AASHTO LRFD Bridge Design Specifications through the use of η_R. The values for η_R provided in that article have been developed in general for the superstructure, and no specific guidance on the application of η_R to foundations is provided. **Paikowsky, et al. (2004)** indicate that a target reliability, β, of 3.0 or more (i.e., an approximate probability of failure of 1 in 1000 or less) is more appropriate for these smaller pile groups that lack redundancy. The η_R factor values recommended in Article 1.3.4 of the AASHTO LRFD Bridge Design Specifications are not adequate to address the difference in redundancy, based on the results provided by **Paikowsky, et al. (2004)**. Therefore, if the resistance factors provided in **Table 8-8** are to be applied to non-redundant pile groups (i.e., less than 5 piles in the group), the resistance factor values in the table should be reduced by 20% to reflect a higher target β value. Greater reductions than this should be considered when a single pile supports an entire bridge pier (i.e., an additional 20 percent reduction in the resistance factor to achieve a β value of approximately 3.5). If the resistance factor is decreased in this manner, the η_R factor provided in Article 1.3.4 of the AASHTO LRFD Bridge Design Specifications should not be increased to address the lack of foundation redundancy.

The resistance factors provided for uplift of single piles are generally less than the resistance factors for axial skin friction under compressive loading. This is consistent with past practice that recognizes the skin friction in uplift is generally less than the skin friction under compressive loading, and is also consistent with the statistical calibrations performed in **Paikowsky, et al. (2004)**. Since the reduction in uplift resistance that occurs in tension relative to the skin friction in compression is taken into account through the resistance factor, the calculation of skin friction resistance using a static pile resistance analysis method should not be reduced from what is calculated from the methods provided in **WSDOT GDM Section 8.12.4.7.5**.

If a pile load test(s) is used to determine the uplift resistance of single piles, consideration should be given to how the pile load test results will be applied to all of the production piles. For uplift, the number of pile load tests required to justify a specific resistance factor are the same as that required for determining compression resistance. Therefore, **Table 8-9** should be used to determine the resistance factor that is applicable. Extrapolating the pile load test results to other untested piles as specified in **WSDOT GDM Section 8.12.4.9** does create some uncertainty, since there is not a way to directly verify that the desired uplift resistance has been obtained for each production pile. This uncertainty has not been quantified. Therefore, it is recommended that a resistance factor of not greater than 0.60 be used if an uplift load test is conducted.

Regarding pile drivability analysis, the only source of load is from the pile driving hammer. Therefore, the load factors provided in Section 3 of the AASHTO LRFD Bridge Design Specifications do not apply. In past practice (e.g., **AASHTO 2002**), no load factors were applied to the stresses imparted to the pile top by the pile hammer. Therefore, a load factor of 1.0 should be used for this type of analysis. Generally, either a wave equation analysis or dynamic testing, or both, are used to determine the stresses in the pile resulting from hammer impact forces. Intuitively, the stresses measured during driving using dynamic testing should be more accurate than the stresses estimated using the wave equation analysis without dynamic testing. However, a statistical analysis and calibration using reliability theory has not been conducted as yet, and a recommendation cannot be provided to differentiate between these two methods regarding the load factor to be applied. See **WSDOT GDM Section 8.12.8** for the specific calculation of the pile structural resistance available for analysis of pile drivability. The structural resistance available during driving determined as specified in **WSDOT GDM Section 8.12.8** considers the ability of the pile to handle the transient stresses resulting from hammer impact, considering variations in the materials, pile/hammer misalignment, and variations in the pile straightness and uniformity of the pile head impact surface.

8.9.3 Resistance Factor Considerations for Drilled Shafts

Resistance factors shall be selected based on the method used for determining the nominal shaft resistance. When selecting a resistance factor for shafts in clays or other easily disturbed formations, local experience with the geologic formations and with typical shaft construction practices shall be considered.

The resistance factors in **Table 8-11** were developed using either statistical analysis of shaft load tests combined with reliability theory (**Paikowsky, et al. 2004**), calibration by fitting to allowable stress design (ASD), or both. Where the two approaches resulted in a significantly different resistance factor, engineering judgment was used to establish the final resistance factor, considering the quality and quantity of the available data used in the calibration. The available reliability theory calibrations were conducted for the **Reese and O'Neill (1988)** method, with the exception of shafts in intermediate geo-materials (IGM's), in which case the **O'Neill and Reese (1999)** method was used. In **WSDOT GDM Section 8.13.4**, the **O'Neill and Reese (1999)** method is recommended. See **Allen (2005)** for a more detailed explanation on the development of the resistance factors for shaft foundation design, and the implications of the differences in these two shaft design methods on the selection of resistance factors.

For the statistical calibrations using reliability theory, a target reliability index, β, of 3.0 (an approximate probability of failure of 1 in 1,000) was used. The selection of this target reliability assumes a small amount of redundancy in the foundation system is present, which is typical for shaft groups containing at least 2 to 4 shafts in the group (**Paikowsky, et al., 2004**). For single shafts, less redundancy will be present. The issue of redundancy, or the lack of it, is addressed in Article 1.3.4 of the AASHTO LRFD Bridge Design Specifications through the use of η_R. The values for η_R provided in that article have been developed in general for the superstructure, and no specific guidance on the application of η_R to foundations is provided. The η_R factor values recommended in Article 1.3.4 of the AASHTO LRFD Bridge Design Specifications are not adequate to address the difference in foundation redundancy, based on the results provided by **Paikowsky, et al. (2004)** and others (see also **Allen 2005**). Therefore, if the resistance factors provided in **Table 8-11** are to be applied to a single shaft supporting a bridge pier, for example, the resistance factor values in the table should be reduced by 20% to reflect a higher target β value of 3.5 (an approximate probability of failure of 1 in 5,000) to be consistent with what has been used generally for design of the superstructure. If the resistance factor is decreased in this manner, the η_R factor provided in Article 1.3.4 of the AASHTO LRFD Bridge Design Specifications should not be increased to address the lack of foundation redundancy.

For shaft groups of 5 or more, greater redundancy than what has been assumed for the development of the shaft resistance factors provided in **Table 8-11** is present. For these larger shaft groups, the resistance factors provided for shafts in **Table 8-11** may be increased by up to 20 percent to achieve a reliability index of 2.3.

When installation criteria are established based on a static load test, the potential for site variability should be considered. The number of load tests required should be established based on the characterization of site subsurface conditions by the field and laboratory exploration and testing program. One or more static load tests should be performed per site to justify using the resistance factors in Table 8-9 for drilled shafts installed within the site.

Table 8-9 identifies resistance factors to be used and numbers of tests needed depending on whether the site variability is classified as low, medium, or high. Site variability may be determined based on judgment, or based on the following suggested criteria (**Paikowsky, et al., 2004**):

Step 1. For each identified significant stratum at each boring location, determine the average property value (e.g., SPT value, q_c value, etc.) within the stratum for each boring.

Step 2. Determine the mean and coefficient of variation of the average values for each stratum determined in Step 1.

Step 3. Categorize the site variability as low if the COV is less than 25%, medium of the COV is 25% or more, but less than 40%, and high if the COV is 40% or more.

See **Paikowsky, et al. (2004)** for additional discussion regarding these site variability criteria.

For the specific case of shafts in clay, the resistance factor recommended by **Paikowsky, et al. (2004)** is much lower than the recommendation from **Barker, et al. (1991)**. Since the shaft design method for clay is nearly the same for both the 1988 and 1999 methods, a resistance factor that represents the average of the two resistance factor recommendations is provided in **Table 8-11**. This difference may point to the differences in local geologic formations and local construction practices, pointing to the importance of taking such issues into consideration when selecting resistance factors, especially for shafts in clay.

IGM's are materials that are transitional between soil and rock in terms of their strength and compressibility, such as residual soils, glacial tills, or very weak rock. See **WSDOT GDM Section 8.13.3.1.2** for a more detailed definition of an IGM.

Since the mobilization of shaft base resistance is less certain than side resistance due to the greater deformation required to mobilize the base resistance, a lower resistance factor relative to the side resistance is provided for the base resistance in **Table 8-11**. **O'Neill and Reese (1999)** make further comment that the recommended resistance factor for tip resistance in sand is applicable for conditions of high quality control on the properties of drilling slurries and base cleanout procedures. If high quality control procedures are not used, the resistance factor for the **O'Neill and Reese (1999)** method for tip resistance in sand should be also be reduced. The amount of reduction should be based on engineering judgment.

Shaft compression load test data should be extrapolated to production shafts that are not load tested as specified in **WSDOT GDM Section 8.13.4.4.5**. Since there is no way to verify shaft resistance for the untested production shafts, other than through good construction inspection and visual observation of the soil or rock encountered in each shaft (where it is possible to make such observations), extrapolation of the shaft load test results to the untested production shafts may introduce some uncertainty. Hence, a reduction of the resistance factor used for design relative to the values provided in **Table 8-9** may be warranted. Statistical data are not available to quantify this at this time. A resistance factor somewhere between the resistance factors specified for the static analysis method in **Table 8-11** and the load test resistance factors specified in **Table 8-9** should be used. Historically, resistance factors higher than 0.70 (or their equivalent safety factor in previous practice) have not been used. Therefore, it is recommended that **Table 8-9** be used, but that the resistance factor not be greater than 0.70.

This issue of uncertainty in how the load test are applied to shafts not load tested is even more acute for shafts subjected to uplift load tests, as failure in uplift can be more abrupt than failure in compression. Hence, a resistance factor of 0.60 for the use of uplift load test results is recommended.

8.10 Resistance Factors for Foundation Design – Extreme Event Limit States

Design of foundations at extreme limit event states shall be consistent with the expectation that structure collapse is prevented and that life safety is protected.

8.10.1 Scour

The foundation shall be designed so that the nominal resistance remaining after the scour resulting from the check flood provides adequate pile resistance to support the unfactored Strength Limit States loads with a resistance factor of 1.0. For uplift resistance of piles and shafts, the resistance factor shall be taken as 0.80 or less. The foundation shall resist not only the loads applied from the structure but also any debris loads occurring during the flood event.

The axial nominal strength after scour due to the check flood must be greater than the unfactored deep foundation load for the Strength Limit State loads. A resistance factor of 1.0 should be used provided that the method used to compute the nominal resistance does not exhibit bias that is unconservative. See **Paikowsky, et al. (2004)** regarding bias values for pile resistance prediction methods.

Design for scour is discussed in **Hannigan, et al. (1997)**.

8.10.2 Other Extreme Event Limit States

Resistance factors for extreme event limit states, including the design of foundations to resist earthquake, ice, vehicle or vessel impact loads, shall be taken as 1.0, with the exception of sliding and bearing resistance of footing foundations. Since the load factor used for the seismic lateral earth pressure for EQ is currently 1.0, to obtain the same level of safety obtained from the AASHTO Standard Specification design requirements for sliding and bearing, a resistance factor of slightly less than 1.0 is required. For sliding and bearing resistance during seismic loading, a resistance factor of 0.90 should be used. For uplift resistance of piles and shafts, the resistance factor shall be taken as 0.80 or less, to account for the difference between compression skin friction and tension skin friction.

Regarding overall stability of slopes that can affect structures, a resistance factor of 0.9, which is equivalent to a factor of safety of 1.1, should in general be used for the extreme event limit state. **WSDOT GDM Section 6.5.3 and Chapter 7** provide additional information and requirements regarding seismic stability of slopes.

8.11 Spread Footing Design

Figure 8-6 provides a flowchart that illustrates the design process, and interaction required between structural and geotechnical engineers, needed to complete a spread footing design. ST denotes steps usually completed by the Structural Designer, while GT denotes those steps normally completed by the geotechnical designer.

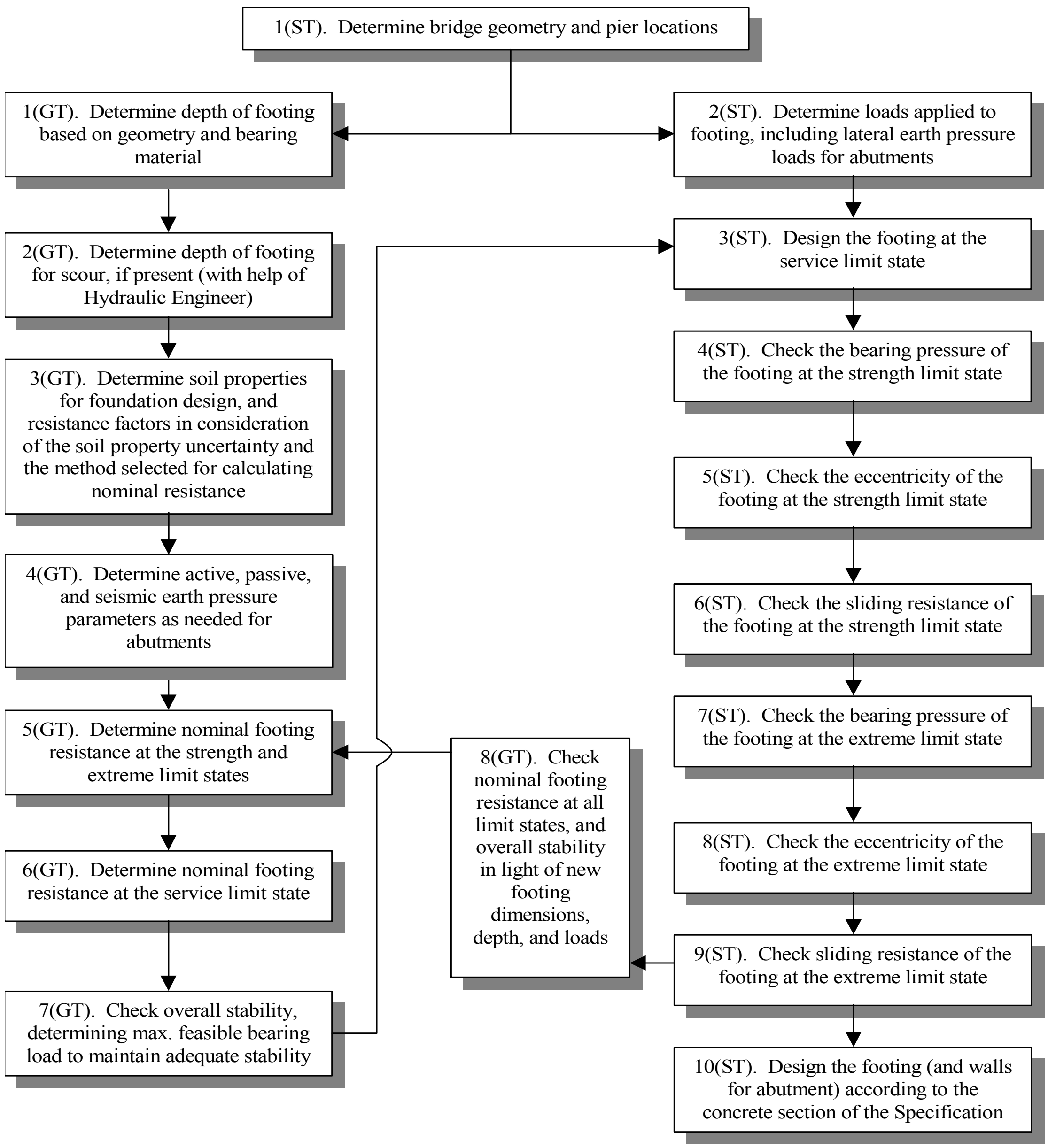


Figure 8-6 Flowchart for LRFD spread footing design.

8.11.1 Loads and Load Factor Application to Footing Design

Figures 8-7 and 8-8 provide definitions and locations of the forces and moments that act on structural footings. Note that the eccentricity used to calculate the bearing stress in geotechnical practice typically is referenced to the centerline of the footing, whereas the eccentricity used to evaluate overturning typically is referenced to point O at the toe of the footing. It is important to not change from maximum to minimum load factors in consideration of the force location relative to the reference point used (centerline of the footing, or point "O" at the toe of the footing), as doing so will cause basic statics to no longer apply, and one will not get the same resultant location when the moments are summed at different reference points. The AASHTO LRFD Bridge design Specifications indicate that the moments should be summed about the center of the footing. **Table 8-12** identifies when to use maximum or minimum load factors for the various modes of failure for the footing (bearing, overturning, and sliding) for each force, for the strength limit state.

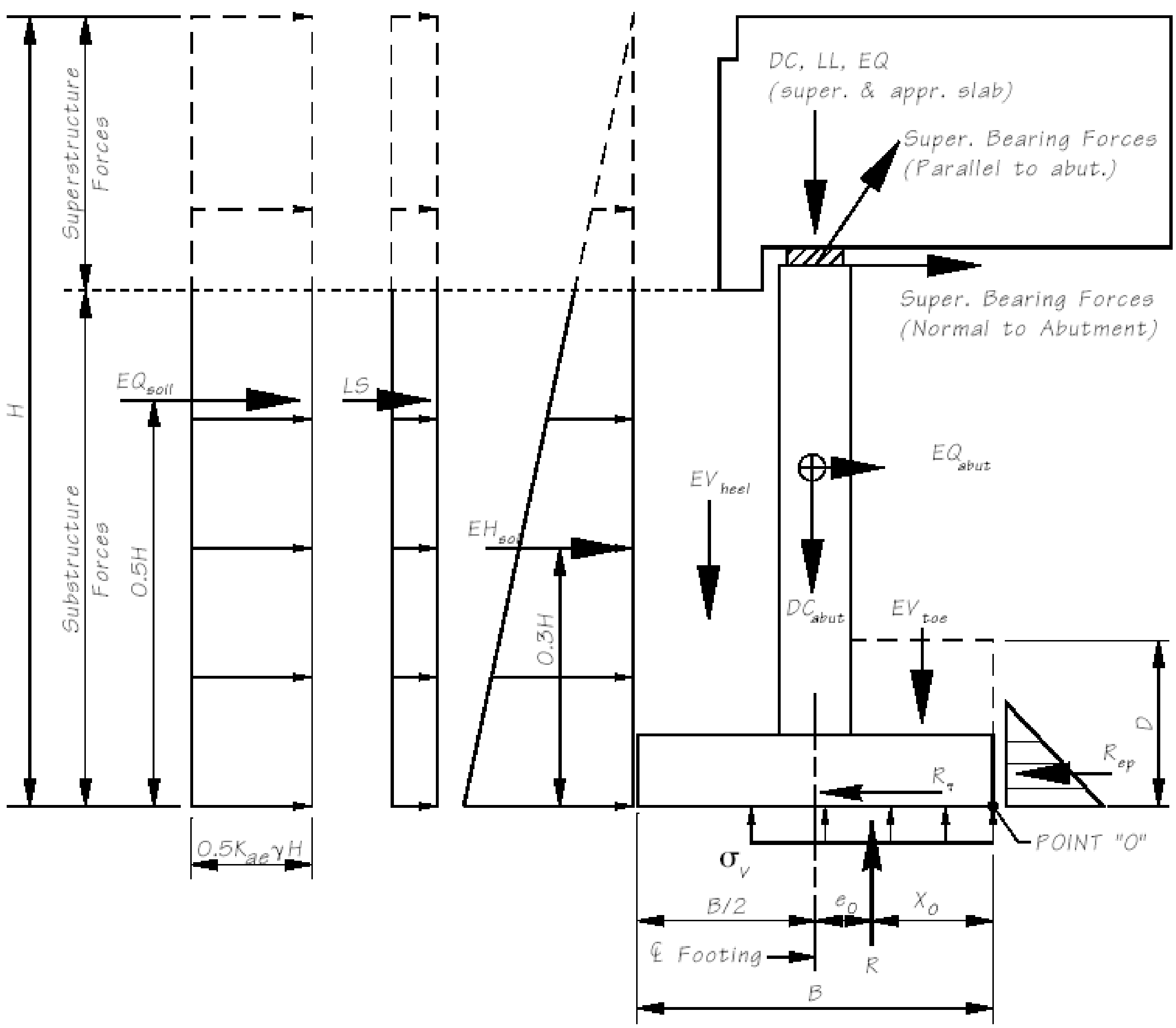


Figure 8-7 Definition and location of forces for stub abutments.

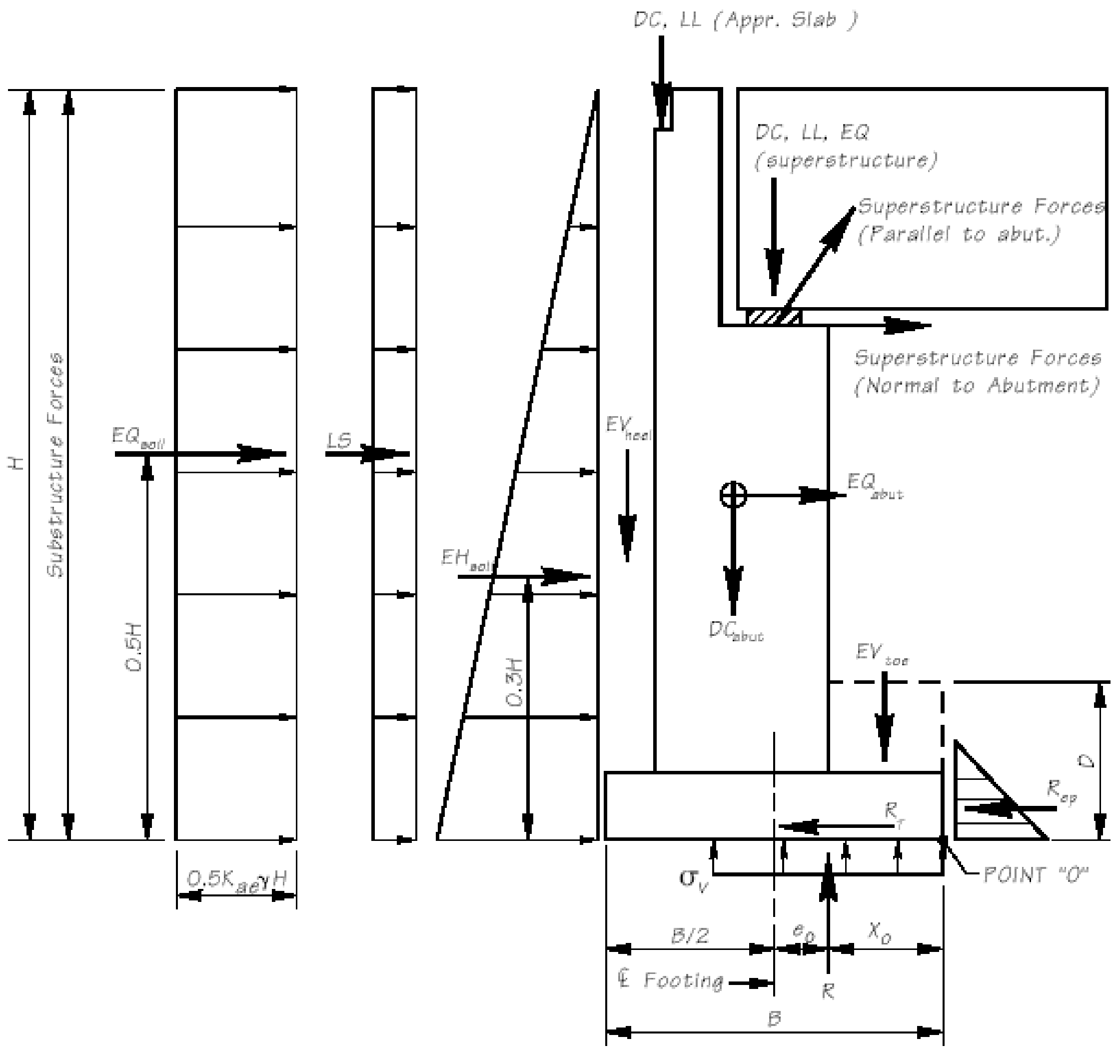


Figure 8-8 Definition and location of forces for L-abutments and interior footings.

The variables shown above in **Figures 8-7 and 8-8** are defined as follows:

DC, LL, EQ	=	vertical structural loads applied to footing/wall (dead load, live load, EQ load, respectively)
DC_{abut}	=	structure load due to weight of abutment
EQ_{abut}	=	abutment inertial force due to earthquake loading
EV_{heel}	=	vertical soil load on wall heel
EV_{toe}	=	vertical soil load on wall toe
EH_{soil}	=	lateral load due to active or at rest earth pressure behind abutment
LS	=	lateral earth pressure load due to live load
EQ_{soil}	=	lateral load due to combined effect of active or at rest earth pressure plus seismic earth pressure behind abutment

R_{ep} = ultimate soil passive resistance (note: height of pressure distribution triangle is determined by the geotechnical engineer and is project specific)

R_τ = soil shear resistance along footing base at soil-concrete interface

σ_V = resultant vertical bearing stress at base of footing

R = resultant force at base of footing

e_o = eccentricity calculated about point O (toe of footing)

X_o = distance to resultant R from wall toe (point O)

B = footing width

H = total height of abutment plus superstructure thickness

	Load Factor		
Load	**Sliding**	**Overturning, e_o**	**Bearing Stress (e_c, σ_V)**
DC, DC_{abut}	Use min. load factor	Use min. load factor	Use max. load factor
LL, LS	Use transient load factor (e.g., LL)	Use transient load factor (e.g., LL)	Use transient load factor (e.g., LL)
EV_{heel}, EV_{toe}	Use min. load factor	Use min. load factor	Use max. load factor
EH_{soil}	Use max. load factor	Use max. load factor	Use max. load factor

Table 8-12 Selection of maximum or minimum spread footing foundation load factors for various modes of failure for the strength limit state.

8.11.2 General Footing Design Considerations

Provisions of this section shall apply to design of isolated, continuous strip and combined footings for use in support of columns, walls and other substructure and superstructure elements.

Special attention shall be given to footings on fill, to make sure that the quality of the fill used below the footing is well controlled and of adequate quality in terms of shear strength and compressibility to support the footing loads. Problems with insufficient bearing and/or excessive settlements in fill can be significant, particularly if poor, e.g., soft, wet, frozen, or nondurable, material is used or if the material is not properly compacted.

Spread footings shall be proportioned and designed such that the supporting soil or rock provides adequate nominal resistance, considering both the potential for adequate bearing strength and the potential for settlement, under all applicable limit states in accordance with the provisions of this section. Spread footings shall be proportioned and located to maintain stability under all applicable limit states, considering the potential for, but not necessarily limited to, overturning (eccentricity), sliding, uplift, overall stability and loss of lateral support.

Spread footings on soil or rock conditions that are determined to be too soft or weak to support the design loads without excessive movement or loss of stability should not be used, unless the unsuitable material can be removed and replaced with suitable and properly compacted engineered fill material, or improved in place, at reasonable cost as compared to other foundation support alternatives.

Footings should be proportioned so that the stress under the footing is as nearly uniform as practicable at the service limit state. The distribution of soil stress should be consistent with properties of the soil or rock and the structure and with established principles of soil and rock mechanics.

8.11.2.1 Footing Bearing Depth

Where the potential for scour, erosion or undermining exists, spread footings shall be located to bear below the maximum anticipated depth of scour, erosion, or undermining as specified in Article 2.6.4.4 of the AASHTO LRFD Bridge Design Specifications.

Spread footings shall be located below the depth of frost potential. Depth of frost potential shall be determined on the basis of local or regional frost penetration data. Consideration may be given to over-excavation of frost susceptible material to below the frost depth and replacement with material that is not frost susceptible.

For footings on slopes, such as at bridge abutments, the footings should be located as shown in the WSDOT LRFD BDM, Section 7.7.1. The footing should also be located to meet the minimum cover requirements provided in WSDOT LRFD BDM, Section 7.7.1.

For spread footings founded on excavated or blasted rock, special attention should be paid to the effect of excavation and/or blasting. Blasting of highly resistant competent rock formations may result in overbreak and fracturing of the rock to some depth below the bearing elevation. Blasting may reduce the resistance to scour within the zone of overbreak or fracturing.

Evaluation of seepage forces and hydraulic gradients should be performed as part of the design of foundations that will extend below the groundwater table. Upward seepage forces in the bottom of excavations can result in piping loss of soil and / or heaving and loss of stability in the base of foundation excavations. Dewatering with wells or wellpoints can control these problems. Dewatering can result in settlement of adjacent ground or structures. If adjacent structures may be damaged by settlement induced by dewatering, seepage cut-off methods such as sheet piling or slurry walls may be necessary.

Consideration should be given to the use of either a geotextile or graded granular filter material to reduce the susceptibility of fine grained material piping into rip rap or open-graded granular foundation material or backfill.

8.11.2.2 Effective Footing Dimensions

For eccentrically loaded footings, a reduced effective area, B' x L', within the confines of the physical footing shall be used in geotechnical design for settlement or bearing resistance. The point of load application shall be at the centroid of the reduced effective area.

The reduced dimensions for an eccentrically loaded rectangular footing shall be taken as:

$$B' = B - 2e_B \quad (8\text{-}2)$$

$$L' = L - 2e_L \quad (8\text{-}3)$$

where:

e_B = eccentricity parallel to dimension B (FT)

e_L = eccentricity parallel to dimension L (FT)

The reduced dimensions for a rectangular footing are shown in **Figure 8-9**.

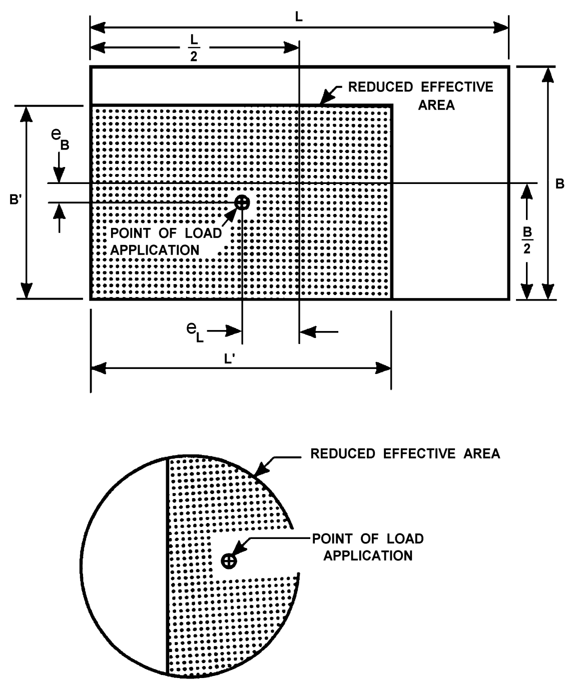


Figure 8-9 Reduced footing dimensions.

For footings that are not rectangular, similar procedures should be used based upon the principles specified above. The reduced effective area of a non-rectangular footing is always concentrically loaded and can be estimated by approximation and judgment. Such an approximation could be made, assuming a reduced rectangular footing size having the same area and centroid as the shaded area of the circular footing shown in **Figure 8-9**.

8.11.2.3 Bearing Stress Distributions

When proportioning footing dimensions to meet settlement and bearing resistance requirements at all limit states, the distribution of bearing stress on the effective area shall be assumed to be:

- Uniform for footings on soils, or
- Linearly varying, i.e., triangular or trapezoidal as applicable, for footings on rock, and shall be calculated as specified in Article 11.6.3.2 of the AASHTO LRFD Bridge Design Specifications.

For structural design of an eccentrically loaded foundation, a triangular or trapezoidal contact stress distribution based on factored loads shall be used for footings bearing on all soil and rock conditions. For purposes of structural design, it is usually assumed that the bearing stress varies linearly across the bottom of the footing. This assumption results in the slightly conservative triangular or trapezoidal contact stress distribution.

8.11.2.4 Inclined Footings on Rock

Footings that are founded on inclined smooth solid rock surfaces and that are not restrained by an overburden of resistant material shall be effectively anchored by means of rock anchors, rock bolts, dowels, keys or other suitable means. Shallow keying of large footings shall be avoided where blasting is required for rock removal. Design of anchorages should include consideration of corrosion potential and protection.

8.11.2.5 Groundwater Effects

Spread footings shall be designed in consideration of the highest anticipated groundwater table. The influences of the groundwater table on the bearing resistance of soils or rocks and on the settlements of the structure shall be considered. In cases where seepage forces are present, they should also be included in the analyses.

8.11.2.6 Nearby Structures

Where foundations are placed adjacent to existing structures, the influence of the existing structure on the behavior of the foundation and the effect of the foundation on the existing structures shall be investigated. Issues to be investigated include, but are not limit to, settlement of the existing structure due to the stress increase caused by the new footing, decreased overall stability due to the additional load created by the new footing, and the effect on the existing structure of excavation, shoring, and/or dewatering to construct the new foundation.

8.11.3 Service Limit State Design of Footings

Service limit state design of spread footings shall include evaluation of total and differential settlement and overall stability. Overall stability of a footing shall be evaluated where:

- Horizontal or inclined loads are present,
- The foundation is placed on embankment,
- The footing is located on, near or within a slope,
- The possibility of loss of foundation support through erosion or scour exists, or
- Bearing strata are significantly inclined.

The design of spread footings is frequently controlled by movement at the service limit state. It is therefore usually advantageous to proportion spread footings at the service limit state and check for adequate design at the strength and extreme event limit states.

Footing foundations shall be designed at the service limit state to meet the tolerable movements for the structure in accordance with **WSDOT GDM Section 8.6.4.1**. The nominal unit bearing resistance at the service limit state, q_{serve}, shall be equal to or less than the maximum bearing stress that that results in settlement that meets the tolerable movement criteria for the structure as calculated in **WSDOT GDM Section 8.11.3.2** and the maximum bearing stress that meets overall stability requirements.

8.11.3.1 Applicable Loads

Immediate settlement shall be determined using load combination Service-I, as specified in Table 3.4.1-1 of the AASHTO LRFD Specifications. Time-dependant settlements in cohesive soils should be determined by using the permanent loads only (i.e., no transient loads).

Various loads may have significant effects on the magnitude of settlements or lateral displacements of the soils. The following factors should be evaluated in the estimation of settlements:

- The ratio of sustained load to total load,
- The duration of sustained loads, and
- The time interval over which settlement or lateral displacement occurs.

The consolidation settlements in cohesive soils are time-dependent; consequently, transient loads have negligible effect. However, in cohesionless soils where the permeability is sufficiently high, elastic deformation of the supporting soil due to transient load can take place. Because deformation in cohesionless soils often takes place during construction while the loads are being applied, it can be accommodated by the structure to an extent, depending on the type of structure and construction method.

Deformation in cohesionless, or granular, soils often occurs as soon as loads are applied. As a consequence, settlements due to transient loads may be significant in cohesionless soils, and they should be included in settlement analyses.

Other factors that may affect settlement, e.g., embankment loading and lateral and/or eccentric loading, and for footings on granular soils, vibration loading from dynamic live loads should also be considered, where appropriate. For guidance regarding settlement due to vibrations, see **Lam and Martin (1986)** or **Kavazanjian, et al., (1997)**.

8.11.3.2 Settlement Analyses

Foundation settlements should be estimated using computational methods based on the results of laboratory or in-situ testing, or both. The soil parameters used in the computations should be chosen to reflect the loading history of the ground, the construction sequence, and the effects of soil layering.

Both total and differential settlements, including time dependant effects, shall be evaluated. Total settlement, including elastic, consolidation, and secondary components, shall be taken as:

$$S_t = S_e + S_c + S_s \qquad (8\text{-}4)$$

where:

Se	=	elastic settlement (FT)
Sc	=	primary consolidation settlement (FT)
Ss	=	secondary settlement (FT)

Elastic, or immediate, settlement is the instantaneous deformation of the soil mass that occurs as the soil is loaded. The magnitude of elastic settlement is estimated as a function of the applied stress beneath a footing or embankment. Elastic settlement is usually small and neglected in design, but where settlement is critical, it is the most important deformation consideration in cohesionless soil deposits and for footings bearing on rock. For footings located on over-consolidated clays, the magnitude of elastic settlement is not necessarily small and should be checked.

In a nearly saturated or saturated cohesive soil, the pore water pressure initially carries the applied stress. As pore water is forced from the voids in the soil by the applied load, the load is transferred to the soil skeleton. Consolidation settlement is the gradual compression of the soil skeleton as the pore water is forced from the voids in the soil. Consolidation settlement is the most important deformation consideration in cohesive soil deposits that possess sufficient strength to safely support a spread footing. While consolidation settlement can occur in saturated cohesionless soils, the consolidation occurs quickly and is normally not distinguishable from the elastic settlement.

Secondary settlement, or creep, occurs as a result of the plastic deformation of the soil skeleton under a constant effective stress. Secondary settlement is of principal concern in highly plastic or organic soil deposits. Such deposits are normally so obviously weak and soft as to preclude consideration of bearing a spread footing on such materials.

The principal deformation component for footings on rock is elastic settlement, unless the rock or included discontinuities exhibit noticeable time-dependent behavior.

The effects of the zone of stress influence, or vertical stress distribution, beneath a footing shall be evaluated in estimating the settlement of the footing. Spread footings bearing on a layered profile consisting of a combination of cohesive soil, cohesionless soil and/or rock shall be evaluated using an appropriate settlement estimation procedure for each layer within the zone of influence of induced stress beneath the footing. The distribution of vertical stress increase below circular (or square) and long rectangular footings, i.e., where $L > 5B$, may be estimated using **Figure 8-10**.

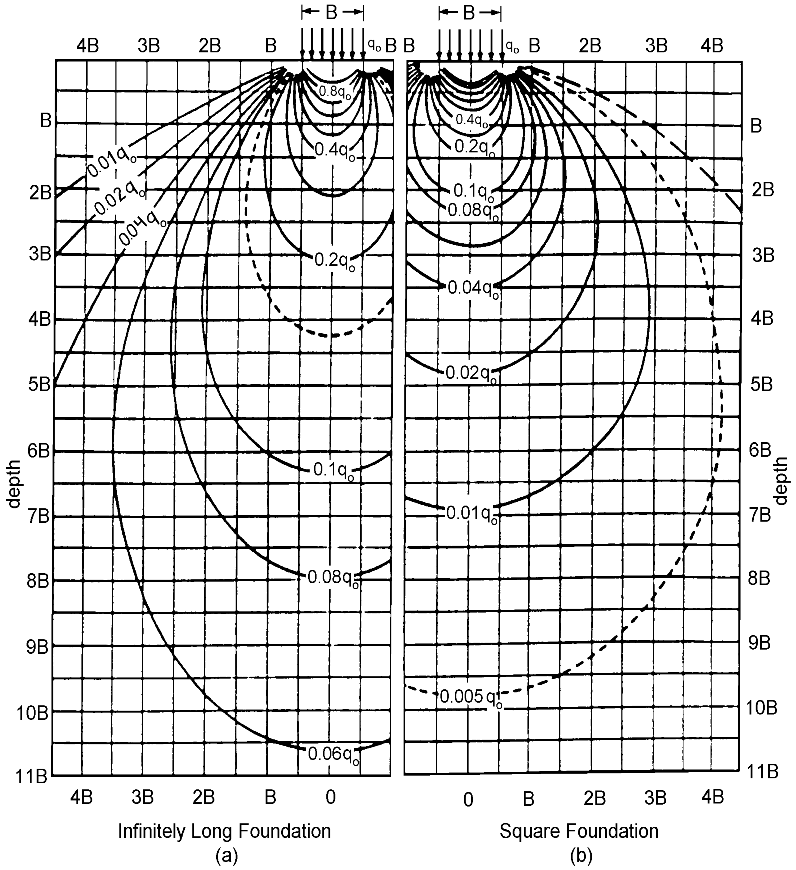


Figure 8-10 Boussinesq vertical stress contours for continuous and square footings modified after Sowers (1979).

For guidance on vertical stress distribution for complex footing geometries, see **Poulos and Davis (1974)** or **Lambe and Whitman (1969)**. Some methods used for estimating settlement of footings on sand include an integral method to account for the effects of vertical stress increase variations. For guidance regarding application of these procedures, see **Gifford et al. (1987)**.

8.11.3.2.1 Settlement of Footings on Cohesionless Soils

Settlements of footings on cohesionless soils shall be estimated using elastic theory or empirical procedures. Although methods are recommended for the determination of settlement of cohesionless soils, experience has indicated that settlements can vary considerably in a construction site, and this variation may not be predicted by conventional calculations.

Settlements of cohesionless soils occur rapidly, essentially as soon as the foundation is loaded. Therefore, the total settlement under the service loads may not be as important as the incremental settlement between intermediate load stages. For example, the total and differential settlement due to loads applied by columns and cross beams is generally less important than the total and differential settlements due to girder placement and casting of continuous concrete decks.

Generally conservative settlement estimates may be obtained using the elastic half-space procedure or the empirical method by **Hough (1959)**. Additional information regarding the accuracy of the methods described herein is provided in **Gifford et al. (1987)** and **Kimmerling (2002)**. This information in combination with local experience and engineering judgment should be used when determining the estimated settlement for a structure foundation, as there may be cases, such as attempting to build a structure grade high to account for the estimated settlement, when overestimating the settlement magnitude could be problematic.

Details of other procedures can be found in textbooks and engineering manuals, including:

- **Terzaghi and Peck 1967**
- **Sowers 1979**
- **U.S. Department of the Navy 1982**
- D'Appolonia (as reported in **Gifford et al. 1987**) – This method includes consideration for over-consolidated sands.
- **Tomlinson 1986**
- **Gifford, et al. 1987**

The elastic half-space approach assumes the footing is flexible and is supported on a homogeneous soil of infinite depth. For general guidance regarding the estimation of elastic settlement of footings on sand, see **Gifford et al. (1987)** and **Kimmerling (2002)**.

The stress distributions used to calculate elastic settlement assume the footing is flexible and supported on a homogeneous soil of infinite depth. The settlement below a flexible footing varies from a maximum near the center to a minimum at the edge equal to about 50 percent and 64 percent of the maximum for rectangular and circular footings, respectively. The settlement profile for rigid footings is assumed to be uniform across the width of the footing. Spread footings of the dimensions normally used for bridges are generally assumed to be rigid, although the actual performance will be somewhere between perfectly rigid and perfectly flexible, even for relatively thick concrete footings, due to stress redistribution and concrete creep.

The elastic settlement of spread footings by the elastic half-space method shall be estimated using **Equation 8-5**.

$$S_e = \frac{\left[q_o\left(1-\nu^2\right)\sqrt{A}\right]}{E_s\beta_z} \tag{8-5}$$

where:

q_o	=	applied vertical stress (KSF)
A	=	area of footing (FT^2)
E_s	=	Young's modulus of soil taken as specified in **WSDOT GDM Chapter 5** if direct measurements of E_s are not available from the results of in situ or laboratory tests (KSF)
β_z	=	shape factor taken as specified in **Table 8-13** (DIM)
ν	=	Poisson's Ratio taken as specified in **WSDOT GDM Chapter 5** if direct measurements of ν are not available from the results of in situ or laboratory tests (DIM)

L/B	Flexible, β_z (average)	β_z Rigid
Circular	1.04	1.13
1	1.06	1.08
2	1.09	1.10
3	1.13	1.15
5	1.22	1.24
10	1.41	1.41

Table 8-13 Elastic Shape and Rigidity Factors, EPRI (1983).

Unless E_S varies significantly with depth, E_S should be determined at a depth of about 1/2 to 2/3 of B below the footing, where B is the footing width. If the soil modulus varies significantly with depth, a weighted average value of E_S should be used.

For footings with eccentric loads, the area, A, should be computed based on reduced footing dimensions as specified in **WSDOT GDM Section 8.11.2.2**.

The accuracy of settlement estimates using elastic theory are strongly affected by the selection of soil modulus and the inherent assumptions of infinite elastic half space. Accurate estimates of soil modulus values are difficult to obtain because the analyses are based on only a single value of soil modulus, and Young's modulus varies with depth as a function of overburden stress. Therefore, in selecting an appropriate value for soil modulus, consideration should be given to the influence of soil layering, bedrock at a shallow depth, and adjacent footings.

Estimation of spread footing settlement on cohesionless soils by the empirical Hough method shall be computed using **Equations 8-6 and 8-7**. The Hough method was developed for normally consolidated cohesionless soils.

SPT blowcounts shall be corrected as specified in **WSDOT GDM Chapter 5** for depth (overburden stress) and hammer efficiency before correlating the N-values to the bearing capacity index, C′.

$$S_e = \sum_{i=1}^{n} \Delta H_i \tag{8-6}$$

in which:

$$\Delta H_i = H_c \frac{1}{C'} \log\left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o}\right) \tag{8-7}$$

where:

n	=	number of soil layers within zone of stress influence of the footing
ΔH_i	=	elastic settlement of layer i (FT)
H_c	=	initial height of layer i (FT)
C'	=	bearing capacity index from **Figure 8-11** (DIM)
σ'_o	=	initial vertical effective stress at the midpoint of layer i (KSF)
$\Delta\sigma_v$	=	increase in vertical stress at the midpoint of layer i (KSF)

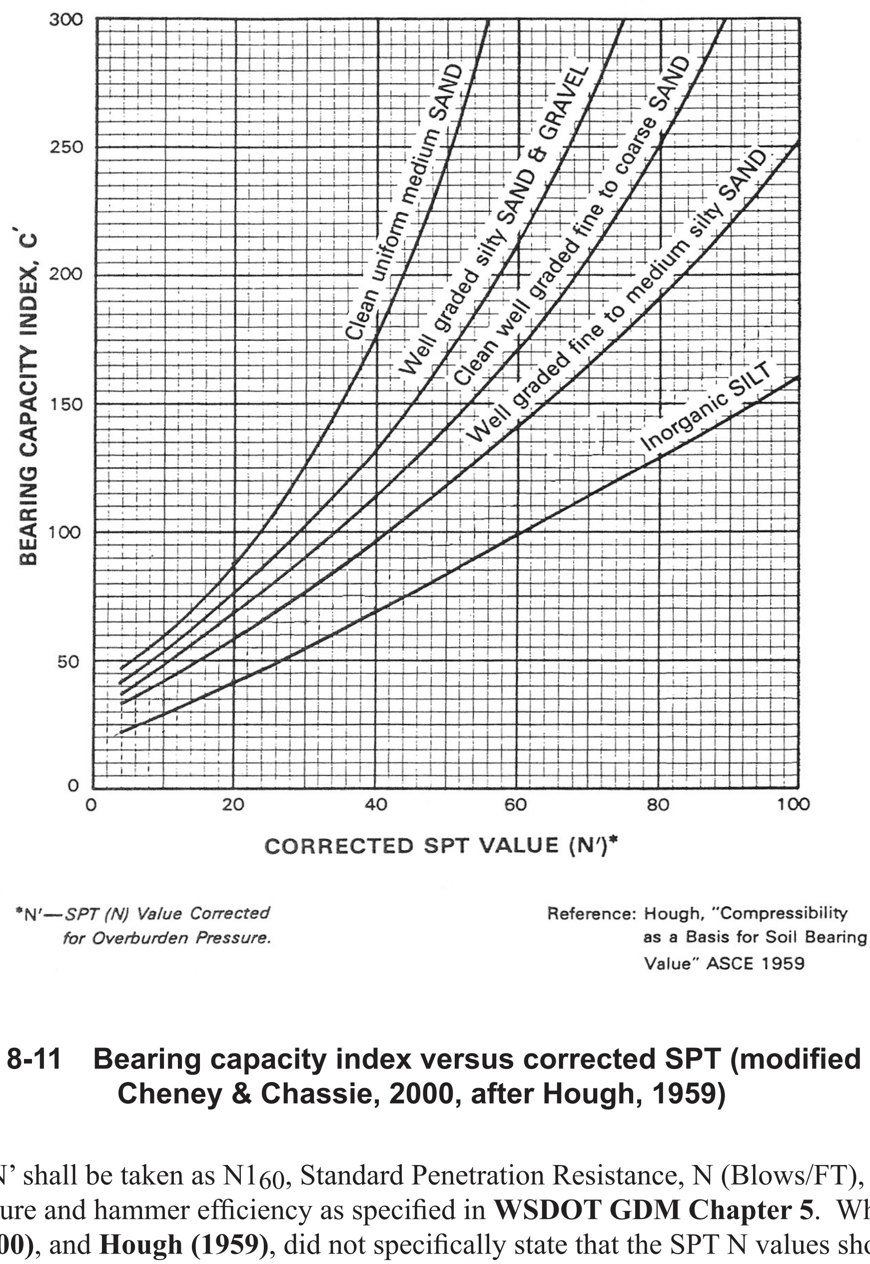


Figure 8-11 Bearing capacity index versus corrected SPT (modified from Cheney & Chassie, 2000, after Hough, 1959)

In **Figure 8-11**, N' shall be taken as $N1_{60}$, Standard Penetration Resistance, N (Blows/FT), corrected for overburden pressure and hammer efficiency as specified in **WSDOT GDM Chapter 5**. While **Cheney and Chassie (2000)**, and **Hough (1959)**, did not specifically state that the SPT N values should be corrected for hammer energy in addition to overburden pressure, due to the vintage of the original work, hammers that typically have an efficiency of approximately 60 percent were in general used to develop the empirical correlations contained in the method. If using SPT hammers with efficiencies that differ significantly from this 60 percent value, the N values should also be corrected for hammer energy, in effect requiring that $N1_{60}$ be used.

The Hough method has several advantages over other methods used to estimate settlement in cohesionless soil deposits, including express consideration of soil layering and the zone of stress influence beneath a footing of finite aerial extent. The subsurface soil profile should be subdivided into layers based on stratigraphy to a depth of about three times the footing width. The maximum layer thickness should be about 10 feet.

The Hough method is applicable to cohesionless soil deposits. The "Inorganic SILT" curve should generally not be applied to soils that exhibit plasticity. Based on experience (see also **Kimmerling, 2002**), the Hough method tends to overestimate settlement of dense sands, and underestimate settlement of very loose silty sands and silts. **Kimmerling (2002)** reports the results of full scale studies where on average the Hough Method overestimated settlement by an average factor of 1.8 to 2.0, though some of the specific cases were close to 1.0. This does not mean that estimated settlements by this method can be reduced by a factor of 2.0. However, based on successful WSDOT experience, for footings on sands and gravels with $N1_{60}$ of 20 blows/ft or more, or sands and gravels that are otherwise known to be overconsolidated (e.g., sands subjected to preloading or deep compaction), reduction of the estimated Hough settlement by up to a factor of 1.5 may be considered, provided the geotechnical designer has not used aggressive soil parameters to account for the Hough method's observed conservatism. The settlement characteristics of cohesive soils that exhibit plasticity should be investigated using undisturbed samples and laboratory consolidation tests as prescribed in **WSDOT GDM Section 8.10.3.2.2**.

8.11.3.2.2 Settlement of Footings on Cohesive Soils

Spread footings bearing on, or above the zone of stress influence of, cohesive soils shall be investigated for consolidation settlement. Elastic and secondary settlement shall also be investigated in consideration of the timing and sequence of construction loading and the tolerance of the structure under study to total and differential movements.

Where laboratory test results are expressed in terms of vertical strain, ε_v, the consolidation settlement of footings shall be taken as:

- For overconsolidated soils where $\sigma'_p > \sigma'_o$:

$$S_c = H_c C_{c\varepsilon} \log\left(\frac{\sigma'_f}{\sigma'_p}\right) \tag{8-8}$$

The soil condition depicted in **Figure 8-12** is for an overconsolidated soil where $\sigma'_{vo} < \sigma'_p$.

- For normally consolidated soils where $\sigma'_p = \sigma'_o$:

$$S_c = H_c C_{c\varepsilon} \log\left(\frac{\sigma'_f}{\sigma'_{pc}}\right) \tag{8-9}$$

- For underconsolidated soils where $\sigma'_p < \sigma'_o$:

$$S_c = H_c C_{c\varepsilon} \log\left(\frac{\sigma'_f}{\sigma'_p}\right) \tag{8-10}$$

where:

H_c	=	initial height of compressible soil layer (FT)
$C_{r\varepsilon}$	=	recompression ratio (DIM)
$C_{c\varepsilon}$	=	compression ratio (DIM)
σ_p	=	maximum past vertical effective stress in soil at midpoint of soil layer under consideration (KSF)
σ_o	=	initial vertical effective stress in soil at midpoint of soil layer under consideration (KSF)
σ_f	=	final vertical effective stress in soil at midpoint of soil layer under consideration (KSF)
σ_{pc}	=	current vertical effective stress in soil, not including the additional stress due to the footing loads, at midpoint of soil layer under consideration (KSF)

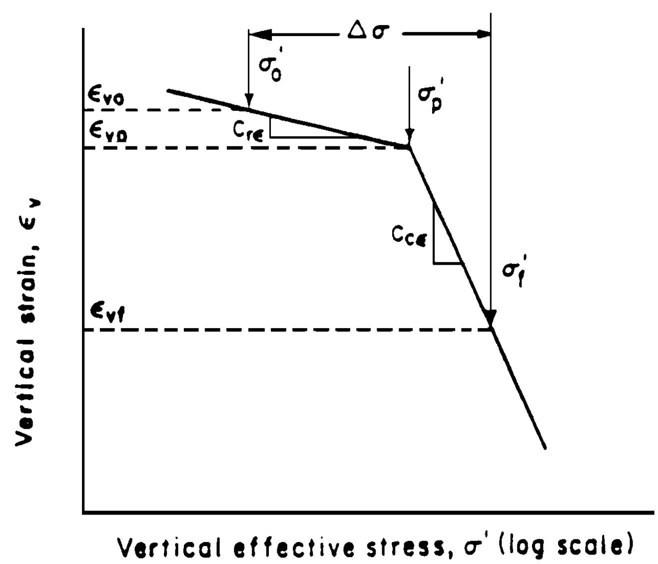


Figure 8-12 Typical consolidation compression curve for overconsolidated soil: vertical Strain versus vertical effective stress (EPRI 1983).

Consolidation settlement may also be estimated using laboratory data expressed in terms of void ratio.

In practice, footings on cohesive soils are most likely founded on overconsolidated clays, and settlements can be estimated using elastic theory (**Baguelin et al. 1978**), or the tangent modulus method (**Janbu 1963, 1967**). Settlements of footings on overconsolidated clay usually occur approximately one order of magnitude faster than soils without preconsolidation, and it is reasonable to assume that they take place as rapidly as the loads are applied. Infrequently, a layer of cohesive soil may exhibit a preconsolidation stress less than the calculated existing overburden stress. The soil is then said to be underconsolidated because a state of equilibrium has not yet been reached under the applied overburden stress. Such a condition may have been caused by a recent lowering of the groundwater table. In this case, consolidation settlement will occur due to the additional load of the structure and the settlement that is occurring to reach a state of equilibrium. The total consolidation settlement due to these two components can be estimated by **Equation 8-10**.

Normally consolidated and underconsolidated soils should be considered unsuitable for direct support of spread footings due to the magnitude of potential settlement, the time required for settlement, for low shear strength concerns, or any combination of these design considerations. Preloading may be considered to mitigate these concerns.

To account for the decreasing stress with increased depth below a footing and variations in soil compressibility with depth, the compressible layer should be divided into vertical increments (i.e., typically 5.0 to 10.0 FT for most normal width footings for transportation applications) and the consolidation settlement of each increment analyzed separately. The total value of S_c is the summation of S_c for each increment.

The magnitude of consolidation settlement depends on the consolidation properties of the soil. These properties include the compression and recompression constants, $C_{c\varepsilon}$, and $C_{r\varepsilon}$; the preconsolidation stress, σ'_p; the current, initial vertical effective stress, σ'_o; and the final vertical effective stress after application of additional loading, σ'_f. An overconsolidated soil has been subjected to larger stresses in the past than at present. This could be a result of preloading by previously overlying strata, desiccation, groundwater lowering, glacial overriding or an engineered preload. If $\sigma'_o = \sigma'_p$, the soil is normally consolidated. Because the recompression constant is typically about an order of magnitude smaller than the compression constant, an accurate determination of the preconsolidation stress, σ'_p, is needed to make reliable estimates of consolidation settlement.

The reliability of consolidation settlement estimates is also affected by the quality of the consolidation test sample and by the accuracy with which changes in σ'_p with depth are known or estimated. As shown in **Figure 8-13**, the slope of the ε versus log σ'_v curve and the location of σ'_p can be strongly affected by the quality of samples used for the laboratory consolidation tests. In general, the use of poor quality samples will result in an overestimate of consolidation settlement. Therefore, the effects of sample disturbance on consolidation parameters should be evaluated and taken into account. Typically, the value of σ'_p will vary with depth as shown in **Figure 8-14**. If the variation of σ'_p with depth is unknown (e.g., only one consolidation test was conducted in the soil profile), actual settlements could be higher or lower than the computed value based on a single value of σ'_p.

The cone penetrometer test may be used to improve understanding of both soil layering and variation of σ'_p with depth by correlation to laboratory tests from discrete locations.

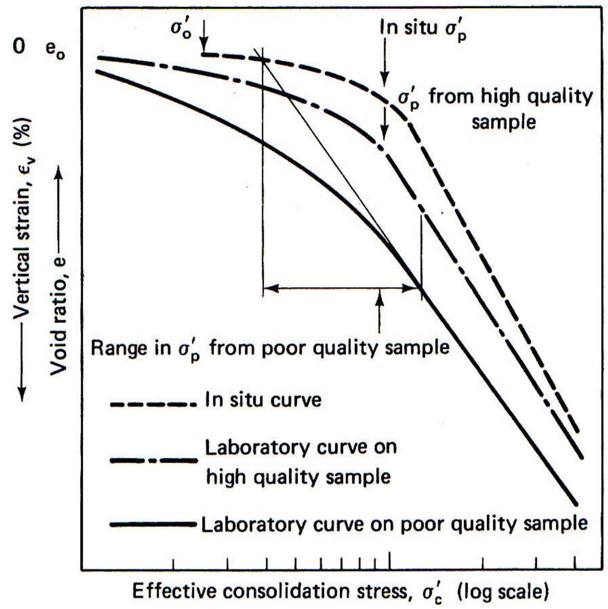


Figure 8-13 Effects of Sample Quality on Consolidation Test Results, Holtz & Kovacs (1981).

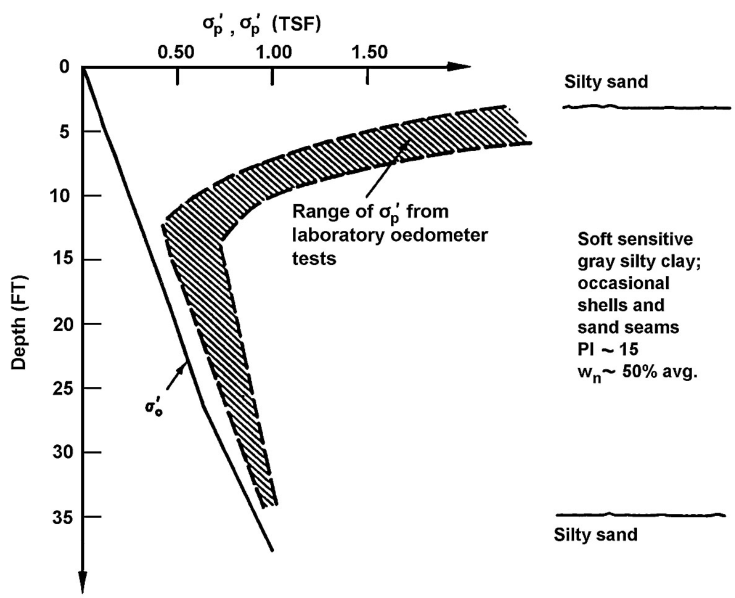


Figure 8-14 Typical Variation of Preconsolidation Stress with Depth (Holtz & Kovacs 1981).

If the footing width is small relative to the thickness of the compressible soil, the effect of three-dimensional loading shall be evaluated and shall be taken as:

$$S_{c(3-D)} = \mu_c S_{c(1-D)} \qquad (8\text{-}11)$$

where:

μ_c	=	reduction factor taken as specified in **Figure 8-15** (DIM)
$S_{c(1-D)}$	=	single dimensional consolidation settlement (FT)

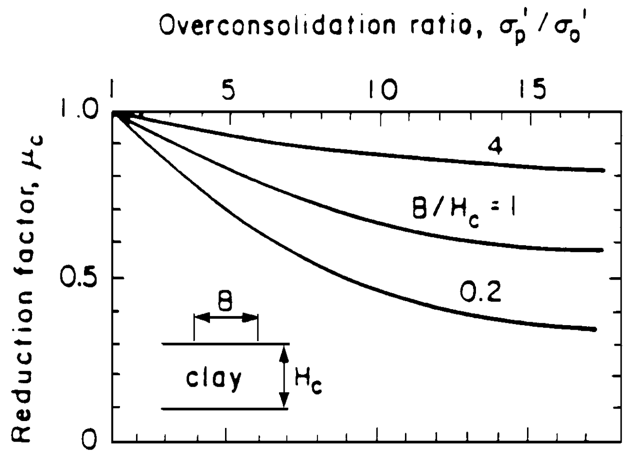


Figure 8-15 Reduction factor to account for effects of three-dimensional consolidation settlement (EPRI 1983).

The time, t, to achieve a given percentage of the total estimated one-dimensional consolidation settlement shall be taken as:

$$t = \frac{TH_v^{\,2}}{c_v} \qquad (8\text{-}12)$$

where:

T	=	time factor taken as specified in **Figure 8-16** (DIM)
H_v	=	length of longest drainage path in compressible layer under consideration (FT)
c_v	=	coefficient of consolidation (FT^2/YR)

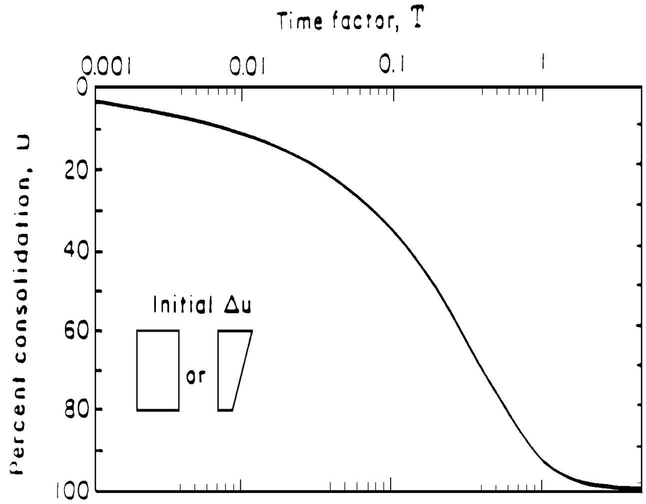


Figure 8-16 Percentage of Consolidation as a Function of Time Factor, T (EPRI 1983).

Consolidation occurs when a saturated compressible layer of soil is loaded and water is squeezed out of the layer. The time required for the (primary) consolidation process to end will depend on the permeability of the soil. Because the time factor, T, is defined as logarithmic, the consolidation process theoretically never ends. The practical assumption is usually made that the additional consolidation past 90% or 95% consolidation is negligible, or is taken into consideration as part of the total long term settlement. Refer to **Winterkorn and Fang (1975)** for values of T for excess pore pressure distributions other than indicated in **Figure 8-16**.

The length of the drainage path is the longest distance from any point in a compressible layer to a drainage boundary at the top or bottom of the compressible soil unit. Where a compressible layer is located between two drainage boundaries, H_V equals one-half the actual height of the layer. Where a compressible layer is adjacent to an impermeable boundary (usually below), H_V equals the full height of the layer.

Computations to predict the time rate of consolidation based on the result of laboratory tests generally tend to over-estimate the actual time required for consolidation in the field. This over-estimation is principally due to:

- The presence of thin drainage layers within the compressible layer that are not observed from the subsurface exploration nor considered in the settlement computations,
- The effects of three-dimensional dissipation of pore water pressures in the field, rather than the one-dimensional dissipation that is imposed by laboratory oedometer tests and assumed in the computations, and
- The effects of sample disturbance, which tend to reduce the permeability of the laboratory tested samples.

If the total consolidation settlement is within the serviceability limits for the structure, the time rate of consolidation is usually of lesser concern for spread footings. If the total consolidation settlement exceeds the serviceability limitations, superstructure damage will occur unless provisions are made for timing of closure pours as a function of settlement, simple support of spans and/or periodic jacking of bearing supports.

Where laboratory test results are expressed in terms of vertical strain, ε_V, the secondary settlement of footings on cohesive soils shall be taken as:

$$S_s = C_{\alpha\varepsilon} H_c \log\left(\frac{t_2}{t_1}\right) \qquad (8\text{-}13)$$

where:

H_c = initial height of compressible soil layer (FT)

t_1 = time when secondary settlement begins, i.e., typically at a time equivalent to 90 percent average degree of consolidation (YR)

t_2 = arbitrary time that could represent the service life of the structure (YR)

$C_{\alpha\varepsilon}$ = modified secondary compression index estimated from the results of laboratory consolidation testing of undisturbed soil samples (DIM)

Secondary compression component of settlement results from compression of bonds between individual clay particles and domains, as well as other effects on the microscale that are not yet clearly understood (**Holtz & Kovacs, 1981**). Secondary settlement is most important for highly plastic clays and organic and micaceous soils. Accordingly, secondary settlement predictions should be considered as approximate estimates only.

If secondary compression is estimated to exceed serviceability limitations, either deep foundations or ground improvement should be considered to mitigate the effects of secondary compression. Experience indicates preloading and surcharging may not be effective in eliminating secondary compression.

8.11.3.2.3 Settlement of Footings on Rock

For footings bearing on fair to very good rock, according to the Geomechanics Classification system, as defined in **WSDOT GDM Chapter 5**, and designed in accordance with the provisions of this section, elastic settlements may generally be assumed to be less than 0.5 IN. When elastic settlements of this magnitude are unacceptable or when the rock is not competent, an analysis of settlement based on rock mass characteristics shall be made.

Where rock is broken or jointed (relative rating of 10 or less for RQD and joint spacing), the rock joint condition is poor (relative rating of 10 or less) or the criteria for fair to very good rock are not met, a settlement analysis should be conducted, and the influence of rock type, condition of discontinuities, and degree of weathering shall be considered in the settlement analysis.

The elastic settlement of footings on broken or jointed rock should be taken as:

- For circular (or square) footings,

$$\rho = q_o\left(1-\nu^2\right)\frac{rI_p}{E_m} \qquad (8\text{-}14)$$

in which:

$$I_p = \frac{\left(\sqrt{\pi}\right)}{\beta_z} \qquad (8\text{-}15)$$

- For rectangular footings;

$$\rho = q_o\left(1-\nu^2\right)\frac{BI_p}{E_m} \qquad (8\text{-}16)$$

in which:

$$I_p = \frac{(L/B)^{1/2}}{\beta_z} \qquad (8\text{-}17)$$

where:

q_o	=	applied vertical stress at base of loaded area (KSF)
ν	=	Poisson's Ratio (DIM)
r	=	radius of circular footing or B/2 for square footing (FT)
I_p	=	influence coefficient to account for rigidity and dimensions of footing (DIM)
E_m	=	rock mass modulus (KSF)
β_z	=	factor to account for footing shape and rigidity (DIM)

Values of I_p should be computed using the β_z values presented in **Table 8-11** for rigid footings. Where the results of laboratory testing are not available, values of Poisson's ratio, ν, for typical rock types may be taken as specified in **WSDOT GDM Chapter 5**. Determination of the rock mass modulus, E_m, should be based on the methods described in **WSDOT GDM Chapter 5**.

The magnitude of consolidation and secondary settlements in rock masses containing soft seams or other material with time-dependent settlement characteristics should be estimated by applying procedures specified in **WSDOT GDM Section 8.10.3.2.2**.

In most cases, it is sufficient to determine settlement using the average bearing stress under the footing.

Where the foundations are subjected to a very large load or where settlement tolerance may be small, settlements of footings on rock may be estimated using elastic theory. The stiffness of the rock mass should be used in such analyses.

The accuracy with which settlements can be estimated by using elastic theory is dependent on the accuracy of the estimated rock mass modulus, E_m. In some cases, the value of E_m can be estimated through empirical correlation with the value of the modulus of elasticity for the intact rock between joints. For unusual or poor rock mass conditions, it may be necessary to determine the modulus from in-situ tests, such as from plate loading and pressuremeter tests.

8.11.3.2.4 Bearing Resistance at the Service Limit State Using Presumptive Values

Bearing resistance estimated using the presumptive allowable bearing pressure for spread footings, if used, shall be a service limit state consideration. Presumptive bearing pressures were developed for use with working stress design. These values may be used for preliminary sizing of foundations, but should generally not be used for final design. If used for final design, presumptive values are only applicable at service limit states.

The use of presumptive values shall be based on knowledge of geological conditions at or near the structure site. Unless more appropriate regional data are available, the presumptive values given in **Table 8-14** may be used. These bearing stresses are settlement limited (e.g., 1 inch) and apply only at the service limit state.

TYPE OF BEARING MATERIAL	CONSISTENCY IN PLACE	BEARING RESISTANCE (KSF)	
		Ordinary Range	Recommended Value of Use
Massive crystalline igneous and metamorphic rock: graphite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)	Very hard, sound rock	120 to 200	160
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)	Hard sound rock	60 to 80	70
Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities	Hard sound rock	30 to 50	40
Weathered or broken bedrock of any kind, except highly argillaceous rock (shale)	Medium hard rock	16 to 24	20
Compaction shale or other highly argillaceous rock in sound condition	Medium hard rock	16 to 24	20
Well-graded mixture of fine- and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very dense	16 to 24	20
Gravel, gravel-sand mixture, boulder-gravel mixtures (GW, GP, SW, SP)	Very dense Medium dense to dense Loose	12 to 20 8 to 14 4 to 12	14 10 6
Coarse to medium sand, and with little gravel (SW, SP)	Very dense Medium dense to dense Loose	8 to 12 4 to 8 2 to 6	8 6 3

Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very dense Medium dense to dense Loose	6 to 10 4 to 8 2 to 4	6 5 3
Fine sand, silty or clayey medium to fine sand (SP, SM, SC)	Very dense Medium dense to dense Loose	6 to 10 4 to 8 2 to 4	6 5 3
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very dense Medium dense to dense Loose	6 to 12 2 to 6 1 to 2	8 4 1
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Very stiff to hard Medium stiff to stiff Soft	4 to 8 2 to 6 1 to 2	6 3 1

Table 8-14 Presumptive bearing resistance for spread footing foundations at the service limit state modified after U.S. Department of the Navy (1982).

Regarding presumptive bearing resistance values for footings on rock, bearing resistance on rock shall be determined using empirical correlation the Geomechanic Rock Mass Rating System, RMR, as specified in **WSDOT GDM Chapter 5**. Local experience should be considered in the use of these semi-empirical procedures.

If the recommended value of presumptive bearing resistance exceeds either the unconfined compressive strength of the rock or the nominal resistance of the concrete, the presumptive bearing resistance shall be taken as the lesser of the unconfined compressive strength of the rock or the nominal resistance of the concrete. The nominal resistance of concrete shall be taken as 0.3 f'_c.

8.11.4 Strength Limit State Design of Footings

The design of spread footings at the strength limit state shall address the following limit states:

- Nominal bearing resistance, considering the soil or rock at final grade, and considering scour as specified in **WSDOT GDM Section 8.6.2**;
- Overturning or excessive loss of contact; and
- Sliding at the base of footing.

8.11.4.1 Bearing Resistance of Footings on Soil

Bearing resistance of spread footings shall be determined based on the highest anticipated position of groundwater level at the footing location. The factored resistance, q_R, at the strength limit state shall be taken as:

$$q_R = \varphi_b \, q_n \qquad (8\text{-}18)$$

where:

φ_b = resistance factor specified in **WSDOT GDM Section 8.9**
q_n = nominal bearing resistance (KSF)

The bearing resistance of footings on soil should be evaluated using soil shear strength parameters that are representative of the soil shear strength under the loading conditions being analyzed. The bearing resistance of footings supported on granular soils should be evaluated for both permanent dead loading conditions and short-duration live loading conditions using effective stress methods of analysis and drained soil shear strength parameters. The bearing resistance of footings supported on cohesive soils should be evaluated for short-duration live loading conditions using total stress methods of analysis and undrained soil shear strength parameters. In addition, the bearing resistance of footings supported on cohesive soils, which could soften and lose strength with time, should be evaluated for permanent dead loading conditions using effective stress methods of analysis and drained soil shear strength parameters.

The position of the groundwater table can significantly influence the bearing resistance of soils through its effect on shear strength and unit weight of the foundation soils. In general, the submergence of soils will reduce the effective shear strength of cohesionless (or granular) materials, as well as the long-term (or drained) shear strength of cohesive (clayey) soils. Moreover, the effective unit weights of submerged soils are about half of those for the same soils under dry conditions. Thus, submergence may lead to a significant reduction in the bearing resistance provided by the foundation soils, and it is essential that the bearing resistance analyses be carried out under the assumption of the highest groundwater table expected within the service life of the structure.

The WSDOT LRFD Bridge Design Manual allows footings to be inclined on slopes of up to 6H: 1V. Footings with inclined bases steeper than this should be avoided wherever possible, using stepped horizontal footings instead. The maximum feasible slope of stepped footing foundations is controlled by the maximum acceptable stable slope for the soil in which the footing is placed. Where use of an inclined footing base must be used, the nominal bearing resistance determined in accordance with the provisions herein should be further reduced using accepted corrections for inclined footing bases in **Munfakh, et al (2001)**.

Where loads are eccentric, the effective footing dimensions, L′ and B′, as specified in **WSDOT GDM Section 8.11.2.2**, shall be used instead of the overall dimensions L and B in all equations, tables and figures pertaining to bearing resistance provided below.

Because the effective dimensions will vary slightly for each limit state under consideration, strict adherence to this provision will require re-computation of the nominal bearing resistance at each limit state. Further, some of the equations for the bearing resistance modification factors based on L and B were not necessarily or specifically developed with the intention that effective dimensions be used. The geotechnical designer must ensure that appropriate values of L and B are used, and that effective footing dimensions L′ and B′ are used appropriately.

Consideration should be given to the relative change in the computed nominal resistance based on effective versus gross footing dimensions for the size of footings typically used for bridges. Judgment should be used in deciding whether the use of gross footing dimensions for computing nominal bearing resistance at the strength limit state would result in a conservative design.

8.11.4.1.1 Theoretical Estimation of Bearing Resistance

The nominal bearing resistance shall be estimated using accepted soil mechanics theories and should be based on measured soil parameters. The soil parameters used in the analyses shall be representative of the soil shear strength under the considered loading and subsurface conditions. The nominal bearing resistance of spread footings on cohesionless soils shall be evaluated using effective stress analyses and drained soil strength parameters. The nominal bearing resistance of spread footings on cohesive soils shall be evaluated using total stress analyses and undrained soil strength parameters. For spread footings bearing on compacted soils, the nominal bearing resistance shall be evaluated using the more critical of either total or effective stress analyses.

The nominal bearing resistance of a soil layer, in TSF, should be determined from the general formulation of **Equation 8-19**, except as noted below. The bearing resistance formulation provided in **Equations 8-19** though **8-22** is the complete formulation as described in the **Munfakh, et al (2001)**. However, in practice, not all of the factors included in these equations have been routinely used.

$$q_n + cN_{cm} + \gamma D_f N_{qm} C_{wa} + 0.5\gamma BN_{ym} C_{wb} \qquad (8\text{-}19)$$

in which:

$$N_{cm} = N_c s_c i_c \qquad (8\text{-}20)$$

$$N_{qm} = N_q s_q d_q i_q \qquad (8\text{-}21)$$

$$N_{\gamma m} = N_\gamma s_\gamma i_\gamma \qquad (8\text{-}22)$$

where:

c = undrained shear strength (KSF)

N_c = cohesion term (undrained loading) bearing capacity factor as specified in **Table 8-15** (DIM)

N_q = surcharge (embedment) term (drained or undrained loading) bearing capacity factor as specified in **Table 8-15** (DIM)

N_γ = unit weight (footing width) term (drained loading) bearing capacity factor as specified in **Table 8-15** (DIM)

γ = total (moist) unit weight of soil above or below the bearing depth of the footing (KCF)

D_f = footing embedment depth (FT)

B = footing width (FT)

C_{wa}, C_{wb} = correction factors to account for the location of the ground water table as specified in **Table 8-16** (DIM)

s_c, s_γ, s_q = footing shape correction factors as specified in **Table 8-17** (DIM)

d_q = correction factor to account for the shearing resistance along the failure surface passing through cohesionless material above the bearing elevation as specified in **Table 8-18** (DIM)

i_c, i_γ, i_q = load inclination factors determined from **equations 8-23 or 8-24, and 8-25 and 8-26** (DIM)

For $\phi = 0$, $i_c = 1 - (nH/cBLN_c)$ (8-23)

For $\phi > 0$, $i_c = i_q - [(1 - i_q)/(N_q - 1)]$ (8-24)

in which:

$$i_q = \left[1 - \frac{H}{(V + cBL\cot\phi)}\right]^n \quad (8\text{-}25)$$

$$i_\gamma = \left[1 - \frac{H}{V + cBL\cot\phi)}\right]^{(n+1)} \quad (8\text{-}26)$$

$$n = [(2 + L/B)/(1 + L/B)]\cos^2\theta + [(2 + B/L)/(1 + B/L)]\sin^2\theta \quad (8\text{-}27)$$

where:

B = footing width (FT)

L = footing length (FT)

H = unfactored horizontal load (KIPS)

V = unfactored vertical load (KIPS)

θ = projected direction of load in the plane of the footing, measured from the side of length L (DEG)

Figure 8-17 shows the convention for determining the θ angle in **Equation 8-27**.

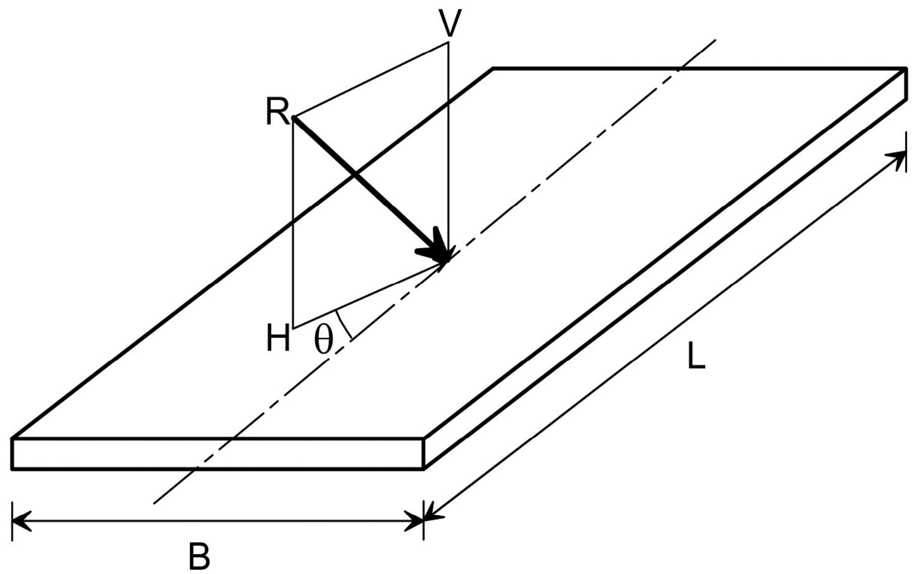


Figure 8-17 Inclined loading conventions.

ϕ	N_c	N_q	N_γ	ϕ	N_c	N_q	N_γ
0	5.14	1.0	0.0	23	18.1	8.7	8.2
1	5.4	1.1	0.1	24	19.3	9.6	9.4
2	5.6	1.2	0.2	25	20.7	10.7	10.9
3	5.9	1.3	0.2	26	22.3	11.9	12.5
4	6.2	1.4	0.3	27	23.9	13.2	14.5
5	6.5	1.6	0.5	28	25.8	14.7	16.7
6	6.8	1.7	0.6	29	27.9	16.4	19.3
7	7.2	1.9	0.7	30	30.1	18.4	22.4
8	7.5	2.1	0.9	31	32.7	20.6	26.0
9	7.9	2.3	1.0	32	35.5	23.2	30.2
10	8.4	2.5	1.2	33	38.6	26.1	35.2
11	8.8	2.7	1.4	34	42.2	29.4	41.1
12	9.3	3.0	1.7	35	46.1	33.3	48.0
13	9.8	3.3	2.0	36	50.6	37.8	56.3
14	10.4	3.6	2.3	37	55.6	42.9	66.2
15	11.0	3.9	2.7	38	61.4	48.9	78.0
16	11.6	4.3	3.1	39	67.9	56.0	92.3
17	12.3	4.8	3.5	40	75.3	64.2	109.4
18	13.1	5.3	4.1	41	83.9	73.9	130.2
19	13.9	5.8	4.7	42	93.7	85.4	155.6
20	14.8	6.4	5.4	43	105.1	99.0	186.5
21	15.8	7.1	6.2	44	118.4	115.3	224.6
22	16.9	7.8	7.1	45	133.9	134.9	271.8

Table 8-15 earing capacity factors N_c (Prandtl, 1921), N_q (Reissner, 1924), and N_γ (Vesic, 1975).

D_w	C_{wa}	C_{wb}
0.0	0.5	0.5
D_f	1.0	0.5
>1.5B+D_f	1.0	1.0

Table 8-16 Coefficients C_{wa} and C_{wb} for various groundwater depths.

Factor	Friction Angle	Cohesion Term (s_c)	Unit Weight Term (s_γ)	Surcharge Term (s_q)
Shape Factors s_c, s_γ, s_q	$\phi = 0$	$1+\left(\frac{B_f}{5L_f}\right)$	1.0	1.0
	$\phi > 0$	$1+\left(\frac{B_f}{L_f}\right)\left(\frac{N_q}{N_c}\right)$	$1-0.4\left(\frac{B_f}{L_f}\right)$	$1+\left(\frac{B_f}{L_f}\tan\phi\right)$

Table 8-17 Shape correction factors s_c, s_γ, s_q.

Friction Angle, ϕ (degrees)	D_f/B_f	d_q
32	1	1.20
	2	1.30
	4	1.35
	8	1.40
37	1	1.20
	2	1.25
	4	1.30
	8	1.35
42	1	1.15
	2	1.20
	4	1.25
	8	1.30

Table 8-18 Depth correction factor d_q.

Shape factors, s_c, s_γ, s_q, from **Table 8-17**, should not be applied simultaneously with inclined loading factors, i_c, i_γ, and i_q, from **equations 8-23 or 8-24, and 8-25 and 8-26**. The load inclination factors i_c, i_γ, and i_q should be taken as 1.0 when using shape factors.

The depth correction factor, d_q, should be used only when the soils above the footing bearing elevation are as competent as the soils beneath the footing level; otherwise, the depth correction factor should be taken as 1.0. For the case where a depth correction factor other than 1.0 should be used, linear interpolations may be made for friction angles in between those values shown in **Table 8-18**. No information is available to extraploate to dq values for firction angles above or below the range shown in the table.

Most geotechnical engineers nationwide have not used the load inclination factors. This is due, in part, to the lack of knowledge of the vertical and horizontal loads at the time of geotechnical explorations and preparation of bearing resistance recommendations. Furthermore, the basis of the load inclination factors computed by **Equations 8-23 to 8-27** is a combination of bearing resistance theory and small scale load tests on 1 IN wide plates on London Clay and Ham River Sand **(Meyerhof, 1953)**. Therefore, the factors do not take into consideration the effects of depth of embedment. Meyerhof further showed that for footings with a depth of embedment ratio of D/B = 1, the effects of load inclination on bearing resistance are relatively small. The theoretical formulation of load inclination factors were further examined by **Brinch-Hansen (1970)**, with additional modification by **Vesic (1973)** into the form provided in **Equations 8-23 to 8-27**.

It should further be noted that the resistance factors provided in **WSDOT GDM Section 8.9** were derived for vertical loads. The applicability of these resistance factors to design of footings resisting inclined load combinations is not currently known. The combination of the resistance factors and the load inclination factors may be overly conservative for footings with an embedment ratio of approximately D/B = 1 or deeper because the load inclination factors were derived for footings without embedment. In practice, therefore, for footings with modest embedment, consideration may be given to omission of the load inclination factors.

These equations have no theoretical limit on the bearing resistance they predict. However, WSDOT limits the nominal bearing resistance for strength and extreme event limit states to 120 KSF on soil. Values greater than 120 KSF should not be used for foundation design in soil.

8.11.4.1.1(a) Considerations for Punching Shear

If local or punching shear failure is possible, the nominal bearing resistance shall be estimated using reduced shear strength parameters c* and ϕ* in **Equations 8-28 and 8-29**. The reduced shear parameters may be taken as:

$$c^* = 0.67c \qquad (8\text{-}28)$$

$$\phi^* = \tan^{-1}(0.67\tan\phi) \qquad (8\text{-}29)$$

where:

c^* = reduced effective stress soil cohesion for punching shear (KSF)

ϕ^* = reduced effective stress soil friction angle for punching shear (DEG)

Local shear failure is characterized by a failure surface that is similar to that of a general shear failure but that does not extend to the ground surface, ending somewhere in the soil below the footing. Local shear failure is accompanied by vertical compression of soil below the footing and visible bulging of soil adjacent to the footing but not by sudden rotation or tilting of the footing. Local shear failure is a transitional condition between general and punching shear failure. Punching shear failure is characterized by vertical shear around the perimeter of the footing and is accompanied by a vertical movement of the footing and compression of the soil immediately below the footing but does not affect the soil outside the loaded area. Punching shear failure occurs in loose or compressible soils, in weak soils under slow (drained) loading, and in dense sands for deep footings subjected to high loads.

The failure mode for a particular footing depends primarily on the compressibility of the soil and the footing depth. The relationship between footing depth, mode of failure, and relative density for footings in sand is shown in **Figure 8-18**.

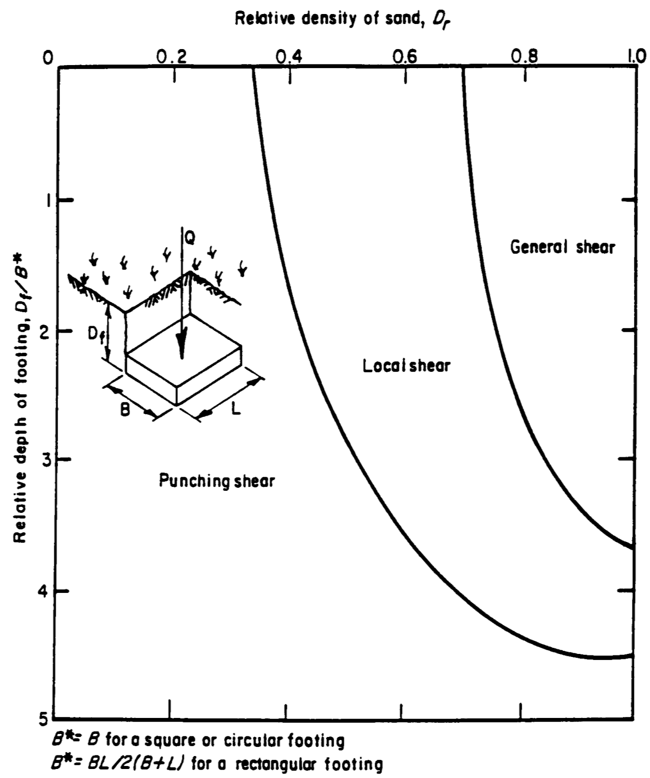


Figure 8-18 Modes of bearing capacity failure for footings in sand.

8.11.4.1.1(b) Considerations for Footings on Slopes

For footings bearing on or near slopes:

N_q = 0.0

In **Equation 8-18**, N_c and N_γ shall be replaced with N_{cq} and $N_{\gamma q}$, respectively, from **Figures 8-19 and 8-20** for footings bearing in or near slopes. In **Figure 8-19**, the slope stability factor, N_s, shall be taken as:

- For $B<H_s$, $N_s = 0$ (8-30)
- For $B \geq H_s$, $N_s = [\gamma H_s/c]$ (8-31)

where:

B = footing width (FT)

H_s = height of sloping ground mass (FT)

A rational numerical approach for determining a modified bearing capacity factor, N_{cq}, for footings on or near a slope that may also be used is given in **Bowles (1988)**.

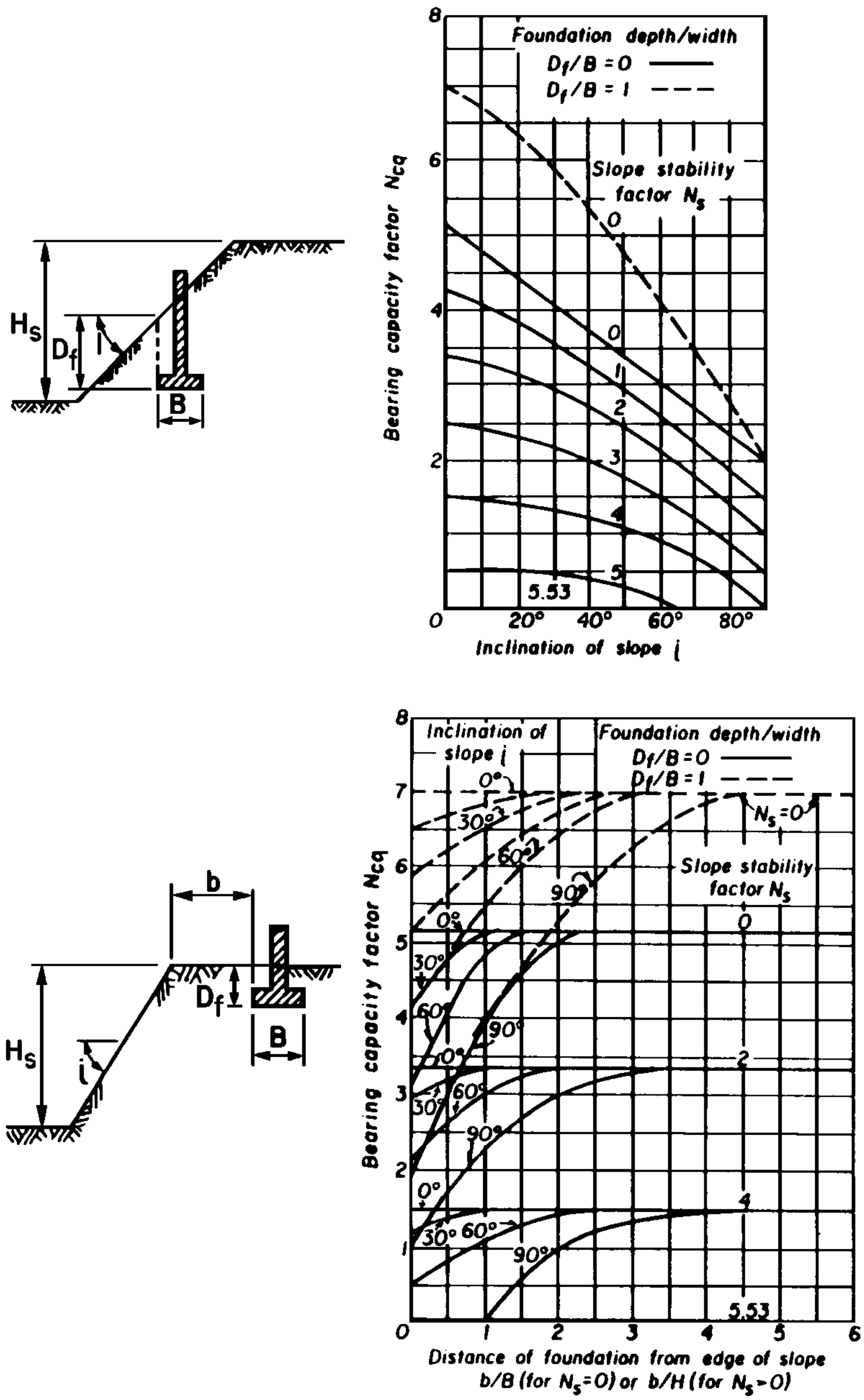


Figure 8-19 Modified bearing capacity factors for footing in cohesive soils and on or adjacent to sloping ground after Meyerhof (1957).

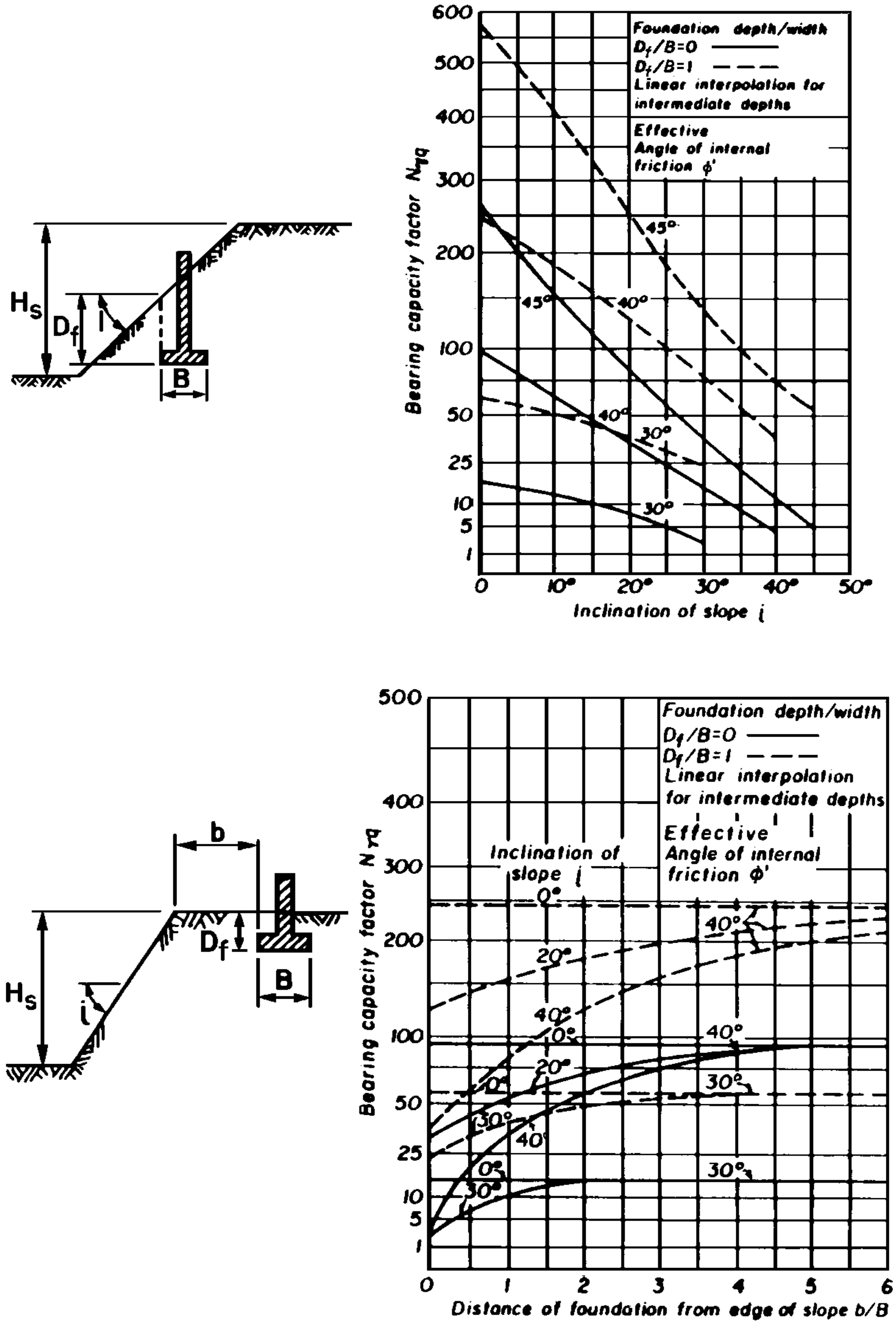


Figure 8-20 Modified bearing capacity factors for footings in cohesionless soils and on or adjacent to sloping ground after Meyerhof (1957).

8.11.4.1.1(c) Considerations for Two Layer Soil Systems – Critical Depth

Where the soil profile contains a second layer of soil with different properties affecting shear strength within a distance below the footing less than H_{crit}, the bearing resistance of the layered soil profile shall be determined using the provisions for two-layered soil systems herein. The distance H_{crit} may be taken as:

$$H_{crit} = \frac{(3B)\ln\left(\frac{q_1}{q_2}\right)}{2\left(1+\frac{B}{L}\right)} \qquad (8\text{-}32)$$

where:

q_1	=	nominal bearing resistance of footing supported in the upper layer of a two-layer system, assuming the upper layer is infinitely thick (KSF)
q_2	=	nominal bearing resistance of a fictitious footing of the same size and shape as the actual footing but supported on surface of the second (lower) layer of a two-layer system (KSF)
B	=	footing width (FT)
L	=	footing length (FT)

8.11.4.1.1(d) Considerations for Two Layer Soil Systems – Undrained Loading

Where a footing is supported on a two-layered soil system subjected to undrained loading, the nominal bearing resistance may be determined using **Equation 8-19** with the following modifications:

c_1	=	undrained shear strength of the top layer of soil as depicted in **Figure 8-21** (KSF)
N_{cm}	=	N_m, a bearing capacity factor as specified below (DIM)
N_{qm}	=	1.0 (DIM)

Where the bearing stratum overlies a stiffer cohesive soil, N_m, may be taken as specified in **Figure 8-22**. Where the bearing stratum overlies a softer cohesive soil, N_m may be taken as:

$$N_m = \left(\frac{1}{\beta_m} + \kappa s_c N_c\right) \le s_c N_c \qquad (8\text{-}33)$$

in which:

$$\beta_m = \frac{BL}{2(B+L)H_{s2}} \qquad (8\text{-}34)$$

$$\kappa = c_2/c_1 \qquad (8\text{-}35)$$

where:

β_m	=	the punching index (DIM)
c_1	=	undrained shear strength of upper soil layer (KSF)
c_2	=	undrained shear strength of lower soil layer (KSF)

H_{s2}	=	distance from bottom of footing to top of the second soil layer (FT)
s_c	=	shape correction factor determined from **Table 8-17**
N_c	=	bearing capacity factor determined herein (DIM)
N_{qm}	=	bearing capacity factor determined herein (DIM)

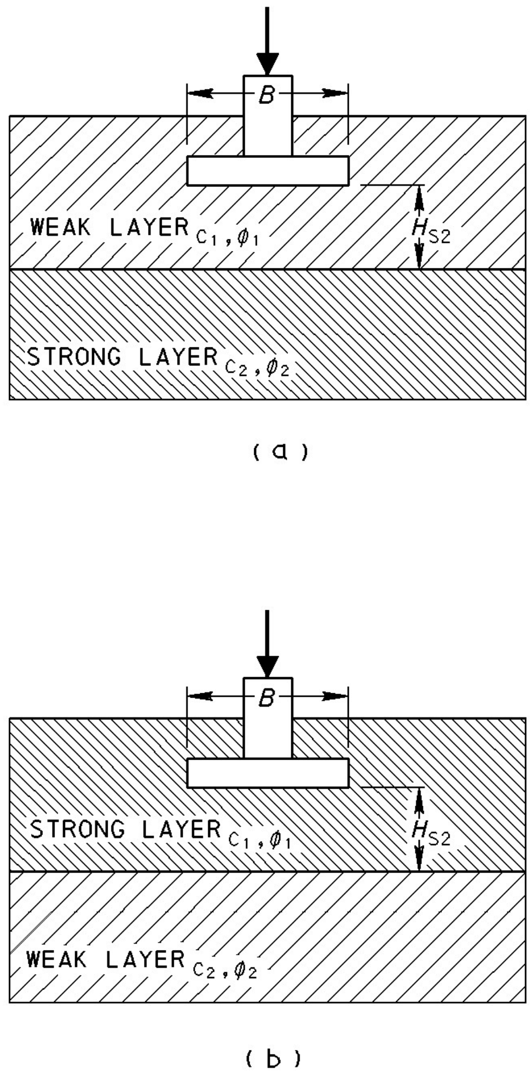


Figure 8-21 Two-layer soil profiles.

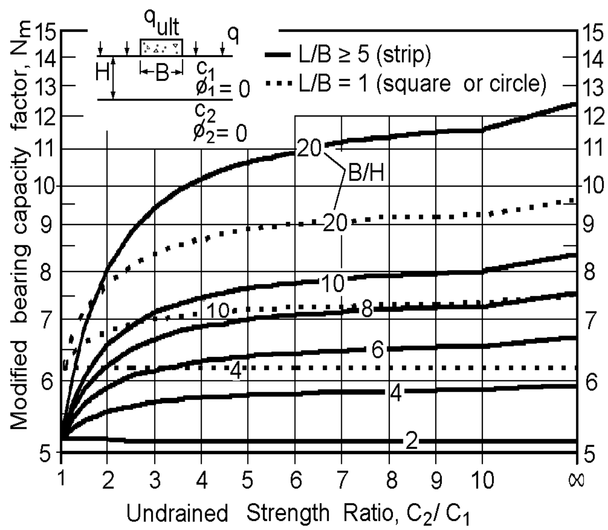


Figure 8-22 Modified bearing factor for two-layer cohesive soil with weaker soil overlying stronger soil (EPRI 1983).

Vesic (1970) developed a rigorous solution for the modified bearing capacity factor, N_m, for the weak undrained layer over strong undrained layer situation that may be used in lieu of the method provided above. This solution is given by the following equation:

$$N_m = \frac{\kappa N_c^*(N_c^* + \beta_m - 1)A}{BC - (\kappa N_c^* + \beta_m - 1))(N_c^* + 1)} \tag{8-36}$$

in which:

$$A = \left[(\kappa + 1) N_c^{*2} + (1 + \kappa\beta_m) N_c^* + \beta_m - 1 \right] \tag{8-37}$$

$$B = \left[\kappa(\kappa + 1) N_c^* + \kappa + \beta_m - 1 \right] \tag{8-38}$$

$$C = \left[(N_c^* + \beta_m) N_c^* + \beta_m - 1 \right] \tag{8-39}$$

For circular or square footings:

$$\beta_m = \frac{B}{4H} \tag{8-40}$$

and $N_c^* = 6.17$

For strip footings:

$$\beta_m = \frac{B}{2H} \tag{8-41}$$

and $N_c^* = 5.14$

8.11.4.1.1(e) Considerations for Two Layer Soil Systems – Drained Loading

Where a footing supported on a two-layered soil system is subjected to a drained loading, the nominal bearing resistance may be taken as:

$$q_n = \left[q_2 + \left(\frac{1}{K}\right)c'_1 \cot\phi'_1\right] e^{2\left[1+\left(\frac{B}{L}\right)\right]K\tan\phi'_1\left(\frac{H}{B}\right)} - \left(\frac{1}{K}\right)c'_1 \cot\phi'_1 \tag{8-42}$$

in which:

$$K = \frac{1-\sin^2\phi'_1}{1+\sin^2\phi'_1} \tag{8-43}$$

where:

c'_1	=	undrained shear strength of the top layer of soil as depicted in **Figure 8-21** (KSF)
q_2	=	nominal bearing resistance of a fictitious footing of the same size and shape as the actual footing but supported on surface of the second (lower) layer of a two-layer system (TSF)
ϕ'_1	=	effective stress angle of internal friction of the top layer of soil (DEG)

If the upper layer is a cohesionless soil and ϕ' equals 25° to 50°, **Equation 8-42** reduces to:

$$q_n = q_2 e^{0.67\left[1+\left(\frac{B}{L}\right)\right]\frac{H}{B}} \tag{8-44}$$

8.11.4.1.2 Semi-Empirical Estimation of Bearing Resistance

The nominal bearing resistance of foundation soils may be estimated from the results of in-situ tests or by observed resistance of similar soils. The use of a particular in-situ test and the interpretation of test results should take local experience into consideration. The following in-situ tests may be used:

- Standard Penetration Test
- Cone Penetration Test

The nominal bearing resistance in sand, in KSF, based on SPT results (**Meyerhof, 1956**) may be taken as:

$$q_n = \frac{\overline{N}1_{60} B}{5}\left(C_{wa}\frac{D_f}{B} + C_{wb}\right) \tag{8-45}$$

where:

$\overline{N}1_{60}$ = Average SPT blow count corrected for both overburden and hammer efficiency effects (Blows/FT) as specified in **WSDOT GDM Chapter 5**. Average the blow count over a depth range from the bottom of the footing to 1.5B below the bottom of the footing.

B = footing width (FT)

C_{wa}, C_{wb} = correction factors to account for the location of the ground water table as specified in **Table 8-16** (DIM)

D_f = footing embedment depth taken to the bottom of the footing (FT)

The nominal bearing resistance, in KSF, for footings on cohesionless soils based on CPT results (**Meyerhof, 1956**) may be taken as:

$$q_n = \frac{q_c B}{40}\left(C_{wa}\frac{D_f}{B} + C_{wb}\right) \qquad (8\text{-}46)$$

where:

q_c = average cone tip resistance within a depth range B below the bottom of the footing (KSF)

B = footing width (FT)

C_{wa}, C_{wb} = correction factors to account for the location of the ground water table as specified in **Table 8-16** (DIM)

D_f = footing embedment depth taken to the bottom of the footing (FT)

In application of these empirical methods, the use of average SPT blowcounts and CPT tip resistances is specified. The resistance factors recommended for bearing resistance included in **Table 8-7** assume the use of average values for these parameters. The use of lower bound values may result in an overly conservative design. However, depending on the availability of soil property data and the variability of the geologic strata under consideration, it may not be possible to reliably estimate the average value of the properties needed for design. In such cases, the engineer may have no choice but to use a more conservative selection of design input parameters to mitigate the additional risks created by potential variability or the paucity of relevant data.

The original derivation of **equations 8-45 and 8-46** by **Meyerhof (1956)** did not include load inclination and other factors addressed by the theoretical approach provided in **WSDOT GDM Section 8.11.4.1.1**. Considering that these equations are empirically based, these other factors should therefore not be considered applicable to **equations 8-45 and 8-46**.

8.11.4.1.3 Plate Load Tests for Determination of Bearing Resistance in Soil

The nominal bearing resistance may be determined by plate load tests, provided that adequate subsurface explorations have been made to determine the soil profile below the foundation. Plate load tests shall be conducted in accordance with AASHTO T 235 and as described in Section 6-02.3(17)D of the WSDOT Standard Specifications for Road, Bridge, and Municipal Construction. The nominal bearing resistance determined from a plate load test may be extrapolated to adjacent footings where the subsurface profile is confirmed by subsurface exploration to be similar.

Plate load tests have a limited depth of influence and furthermore may not disclose the potential for long-term consolidation of foundation soils. Scale effects should be addressed when extrapolating the results to performance of full scale footings. Extrapolation of the plate load test data to a full scale footing should be based on the design procedures provided herein for settlement (service limit state) and bearing resistance (strength and extreme event limit state), with consideration to the effect of the stratification (i.e., layer thicknesses, depths, and properties). Plate load test results should be applied only within a sub-area of the project site for which the subsurface conditions (i.e., stratification, geologic history, properties) are relatively uniform.

8.11.4.2 Bearing Resistance of Footings on Rock

The methods used for design of footings on rock shall consider the presence, orientation, and condition of discontinuities, weathering profiles, and other similar profiles as they apply at a particular site.

For footings on competent rock, reliance on simple and direct analyses based on uniaxial compressive rock strengths and RQD may be applicable. For footings on less competent rock, more detailed investigations and analyses shall be performed to account for the effects of weathering and the presence and condition of discontinuities. The designer shall judge the competency of a rock mass by taking into consideration both the nature of the intact rock and the orientation and condition of discontinuities of the overall rock mass. Where engineering judgment does not verify the presence of competent rock, the competency of the rock mass should be verified using the procedures for RMR rating in **WSDOT GDM Chapter 5**.

The design of spread footings bearing on rock is frequently controlled by either overall stability, i.e., the orientation and conditions of discontinuities, or load eccentricity considerations. The geotechnical designer should verify adequate overall stability at the service limit state and size the footing based on eccentricity requirements at the strength limit state before checking nominal bearing resistance at both the service and strength limit states.

8.11.4.2.1 Semi-Empirical Methods for Bearing on Rock

The nominal bearing resistance of rock should be determined using empirical correlation with the Geomechanics Rock Mass Rating system. Local experience shall be considered in the use of these semi-empirical procedures. The factored bearing stress of the foundation shall not be taken to be greater than the factored compressive resistance of the footing concrete.

The bearing resistance of jointed or broken rock may be estimated using the semi-empirical procedure developed by **Carter and Kulhawy (1988)**. This procedure is based on the unconfined compressive strength of the intact rock core sample. Depending on rock mass quality measured in terms of RMR system, the nominal bearing resistance of a rock mass varies from a small fraction to six times the unconfined compressive strength of intact rock core samples.

8.11.4.2.2 Analytic Method for Bearing on Rock

The nominal bearing resistance of foundations on rock shall be determined using established rock mechanics principles based on the rock mass strength parameters. The influence of discontinuities on the failure mode shall also be evaluated.

Depending upon the relative spacing of joints and rock layering, bearing capacity failures for foundations on rock may take several forms. Except for the case of a rock mass with closed joints, the failure modes are different from those in soil. Procedures for estimating bearing resistance for each of the failure modes can be found in **Kulhawy and Goodman (*1987*), Goodman (*1989*), and Sowers (*1979*)**.

8.11.4.2.3 Load Test for Bearing on Rock

Where appropriate, load tests may be performed to determine the nominal bearing resistance of foundations on rock.

8.11.4.3 Strength Limit State Design of Footings for Load Eccentricity

The eccentricity of loading at the strength limit state, evaluated based on factored loads shall not exceed:

- One-fourth (1/4) of the corresponding footing dimension, B or L, for footings on soils, or
- Three-eighths (3/8) of the corresponding footing dimensions B or L, for footings on rock.

Note that a comprehensive parametric study was conducted for cantilevered retaining walls of various heights and soil conditions. The base widths obtained using the LRFD load factors and eccentricity of B/4 were comparable to those of ASD with an eccentricity of B/6.

8.11.4.4 Design of Footings to Resist Failure by Sliding

Failure by sliding shall be investigated for footings that support horizontal or inclined load and/or are founded on slopes.

For foundations on clay soils, the possible presence of a shrinkage gap between the soil and the foundation shall be considered. If passive resistance is included as part of the shear resistance required for resisting sliding, consideration shall also be given to possible future removal of the soil in front of the foundation.

The factored resistance against failure by sliding, in KIPS, shall be taken as:

$$R_n = \varphi R_n = \varphi_\tau R_\tau + \varphi_{ep} R_{ep} \tag{8-47}$$

where:

R_n = nominal sliding resistance against failure by sliding (KIPS)

φ_τ = resistance factor for shear resistance between soil and foundation specified in **Table 8-7**

R_τ = nominal sliding resistance between soil and foundation (KIPS)

φ_{ep} = resistance factor for passive resistance specified in **Table 8-7**

R_{ep} = nominal passive resistance of soil available throughout the design life of the structure (KIPS)

If the soil beneath the footing is cohesionless, then:

$R_\tau = V \tan \delta$ (8-48)

for which:

$\tan \delta$	=	$\tan \phi_f$ for concrete cast against soil
	=	$0.8 \tan \phi_f$ for precast concrete footing
ϕ_f	=	internal friction angle of drained soil (DEG), and
V	=	total vertical force (KIPS)

Rough footing bases usually occur where footings are cast in-situ. Precast concrete footings may have smooth bases.

For footings that rest on clay, the sliding resistance should be taken as the lesser of:

- The cohesion of the clay, or
- Where footings are supported on at least 6.0 IN of compacted granular material, one-half the normal stress on the interface between the footing and soil, as shown in **Figure 8-23** for retaining walls.

The following notation shall be taken to apply to **Figure 8-23**:

R_τ	=	nominal sliding resistance between soil and foundation (KIPS) expressed as the shaded area under the q_s diagram
q_s	=	unit shear resistance, equal to s_u or $0.5\sigma'_v$, whichever is less
s_u	=	undrained shear strength (KSF)
σ'_v	=	vertical effective stress (KSF)

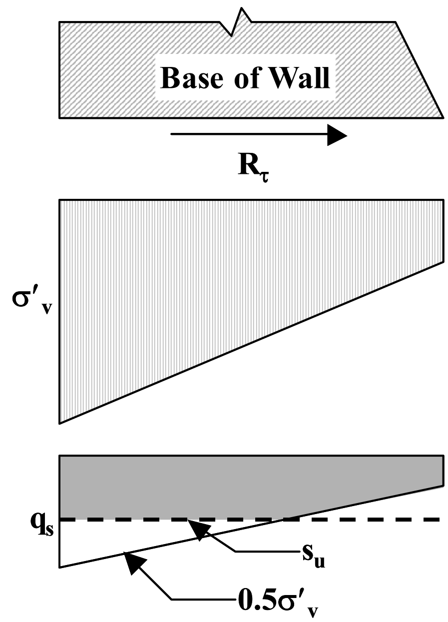


Figure 8-23 Procedure for estimating nominal sliding resistance for walls on clay.

8.11.5 Extreme Event Limit State Design of Footings

Extreme limit state design checks for spread footings shall include, but not necessarily be limited to:

- Bearing resistance
- Eccentric load limitations (overturning)
- Sliding
- Overall stability

Resistance factors shall be as specified in **WSDOT GDM Section 8.10**.

Footings shall not be located on or within liquefiable soil. Footings may be located on liquefiable soils that have been improved through densification or other means so that they do not liquefy. Footings may also be located above liquefiable soil in a non-liquefiable layer if the footing is designed to meet all Extreme Event limit states. In this case, liquefied soil parameters shall be used for the analysis (see **WSDOT GDM Chapter 6**). The footing shall be stable against an overall stability failure of the soil (see **WSDOT GDM Section 8.6.5.2**) and lateral spreading resulting from the liquefaction (see **WSDOT GDM Chapter 6**).

Footings located above liquefiable soil but within a non-liquefiable layer shall be designed to meet the bearing resistance criteria established for the structure for the Extreme Event Limit State. The bearing resistance of a footing located above liquefiable soils shall be determined considering the potential for a punching shear condition to develop, and shall also be evaluated in accordance with **WSDOT GDM Sections 8.11.4.1.1(c), 8.11.4.1.1(d), and 8.11.4.1.1(e)**, assuming the soil to be in a liquefied condition.

Settlement of the liquefiable zone shall also be evaluated to determine if the extreme event limit state criteria for the structure the footing is supporting are met. The **Tokimatsu and Seed (1987) or the Ishihara and Yoshimine (1992)** procedure should be used to estimate settlement.

For footings, whether on soil or on rock, the eccentricity of loading at the extreme limit state shall not exceed one-third (0.33) of the corresponding footing dimension, B or L, for γ_{EQ} = 0.0 and shall not exceed four-tenths (0.40) of the corresponding footing dimension, B or L, for γ_{EQ} = 1.0. If live loads act to reduce the eccentricity for the Extreme Event I limit state, γ_{EQ} shall be taken as 0.0.

8.12 Driven Pile Foundation Design

Figure 8-24 provides a flowchart that illustrates the design process, and interaction required between structural and geotechnical engineers, needed to complete a driven pile foundation design. ST denotes steps usually completed by the Structural Designer, while GT denotes those steps normally completed by the geotechnical designer.

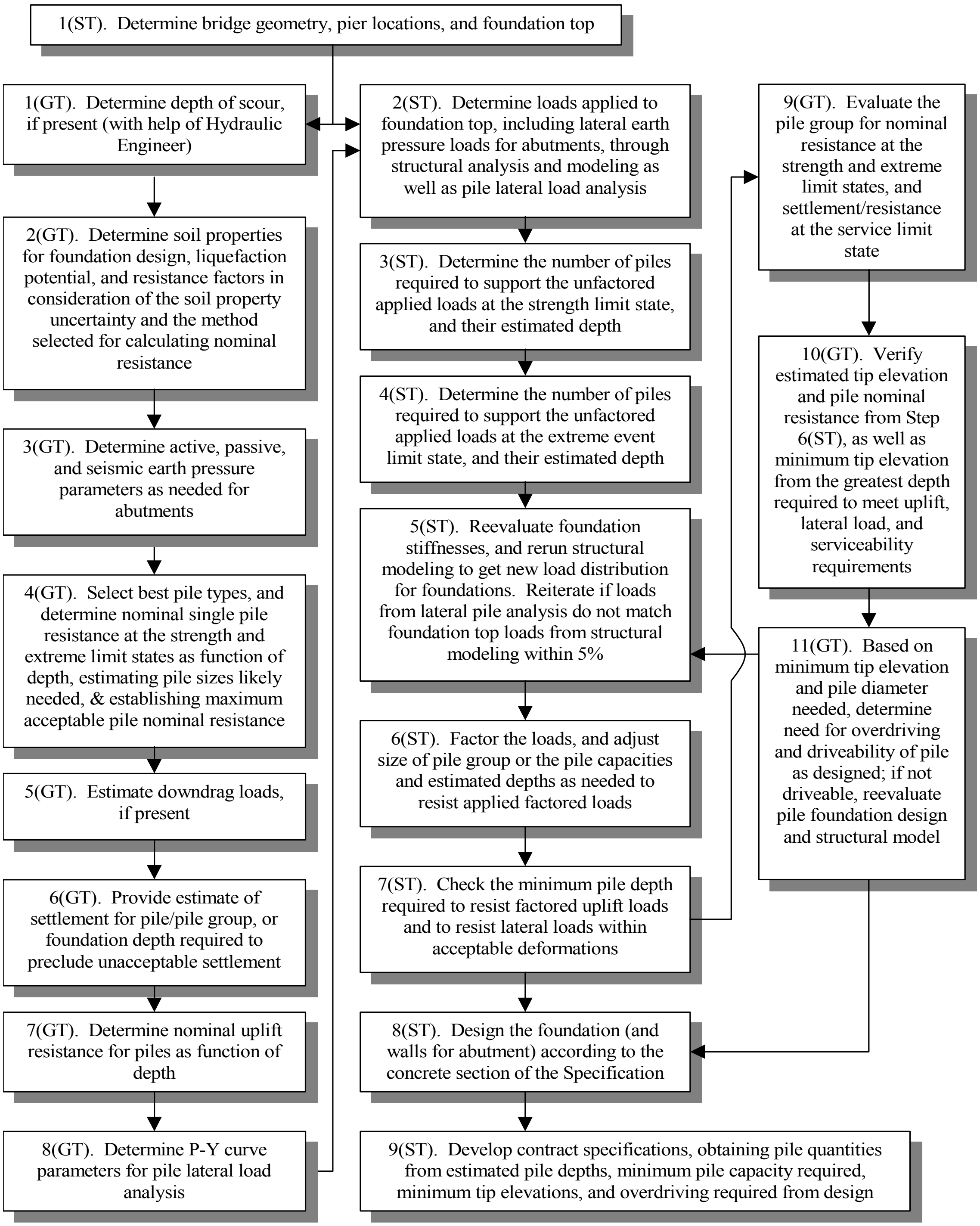


Figure 8-24 Design flowchart for pile foundation design.

8.12.1 Loads and Load Factor Application to Driven Pile Design

Figures 8-25 and 8-26 provide definitions and typical locations of the forces and moments that act on deep foundations such as driven piles. **Table 8-19** identifies when to use maximum or minimum load factors for the various modes of failure for the pile (bearing, uplift, and lateral loading) for each force, for the strength limit state.

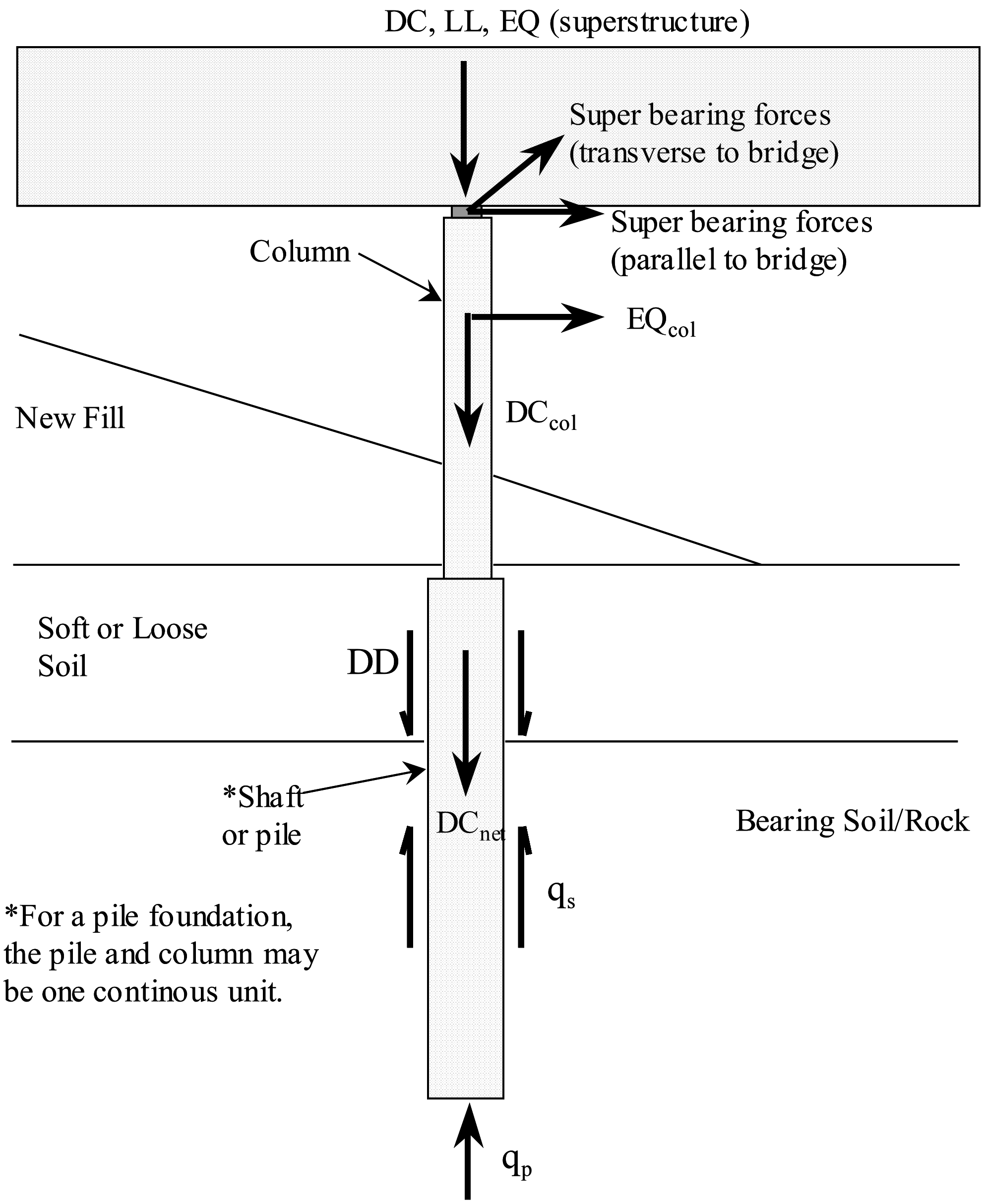


Figure 8-25 Definition and location of forces for integral shaft column or pile bent.

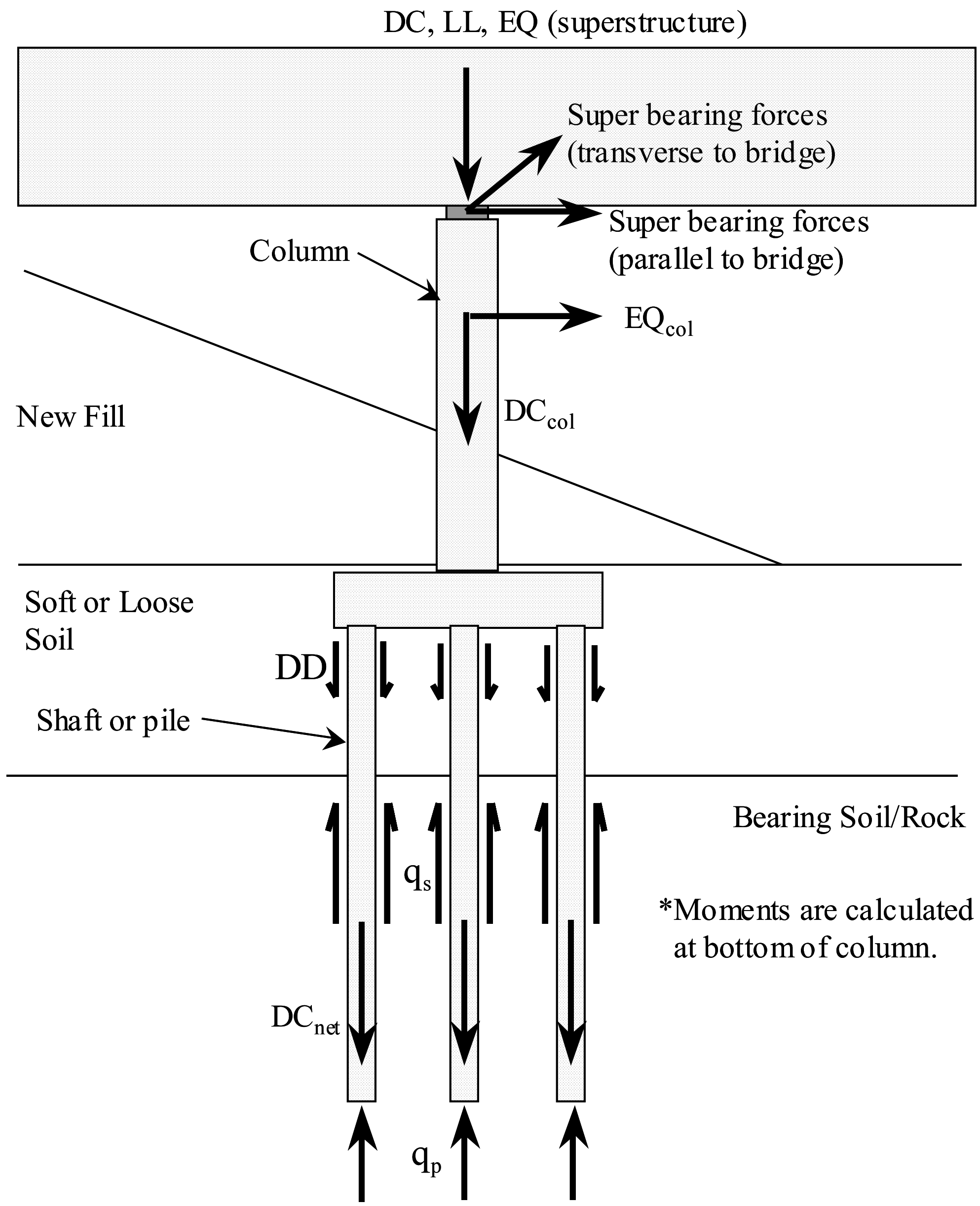


Figure 8-26 Definition and location of forces for pile or shaft supported footing.

where,

DC_{col} = structure load due to weight of column

EQ_{col} = earthquake inertial force due to weight of column

q_p = ultimate end bearing resistance at base of shaft (unit resistance)

q_s = ultimate side resistance on shaft (unit resistance)

DD = ultimate down drag load on shaft (total load)

DC_{net} = unit weight of concrete in shaft minus unit weight of soil times the shaft volume below the groundline (may include part of the column if the top of the shaft is deep due to scour or for other reasons

All other forces are as defined previously.

	Load Factor		
Load	**Bearing Stress**	**Uplift**	***Lateral Loading**
DC, DC_{col}	Use max. load factor	Use min. load factor	Use max load factor
LL	Use transient load factor (e.g., LL)	Use transient load factor (e.g., LL)	Use transient load factor (e.g., LL)
DC_{net}	Use max. load factor	Use min. load factor	N/A
DD	Use max. load factor	Treat as resistance, and use resistance factor for uplift	N/A

*Use unfactored loads to get force distribution in structure, then factor the resulting forces for final structural design.

Table 8-19 Selection of maximum or minimum deep foundation load factors for various modes of failure for the strength limit state.

All forces and load factors are as defined previously.

The loads and load factors to be used in pile foundation design shall be as specified in Section 3 of the AASHTO LRFD Bridge Design Specifications. Computational assumptions that shall be used in determining individual pile loads are described in Section 4 of the AASHTO LRFD Bridge Design Specifications.

8.12.2 General Considerations for Pile Foundation Geotechnical Design

Pile design shall address the following issues as appropriate:

- Nominal axial resistance to be specified in the contract, type of pile, and size of pile group required to provide adequate support, with consideration of how nominal axial pile resistance will be determined in the field.
- Group interaction.
- Pile quantity estimation from estimated pile penetration required to meet nominal axial resistance and other design requirements.

- Minimum pile penetration necessary to satisfy the requirements caused by uplift, scour, downdrag, settlement, liquefaction, lateral loads and seismic conditions.
- Foundation deflection to meet the established movement and associated structure performance criteria.
- Pile foundation nominal structural resistance.
- Verification of pile drivability to confirm that acceptable driving stresses and blow counts can be achieved with an available driving system to meet all contract acceptance criteria.
- Long-term durability of the pile in service (i.e. corrosion and deterioration).

8.12.2.1 Driven Pile Sizes and Maximum Resistances

In lieu of more detailed structural analysis, the general guidance on pile types, sizes, and nominal resistance values provided in **Table 8-20** may be used to select pile sizes and types for analysis. The Geotechnical Division limits the maximum nominal pile resistance for 24 inch piles to 1500 KIPS and 18 inch piles to 1,000 KIPS, and may limit the nominal pile resistance for a given pile size and type driven to a given soil/rock bearing unit based on experience with the given soil/rock unit. The maximum resistance allowed in that given soil/rock unit may be increased by the WSDOT Geotechnical Division per mutual agreement with the Bridge and Structures Office if a pile load test is performed.

	Pile Type and Diameter (in.)			
Nominal pile Resistance (KIPS)	**Closed End Steel Pipe/Cast-in-Place Concrete Piles**	***Precast, Prestressed Concrete Piles**	**Steel H-Piles**	**Timber Piles**
120	-	-	-	See WSDOT Standard Specs.
240	-	-	-	See WSDOT Standard Specs.
330	12 in.	13 in.	-	-
420	14 in.	16 in.	12 in.	-
600	18 in. nonseismic areas, 24 in. seismic areas	18 in.	14 in.	-
900	24 in.	Project Specific	Project Specific	-

*Precast, prestressed concrete piles are generally not used for highway bridges, but are more commonly used for marine work.

Table 8-20 Typical pile types and sizes for various nominal pile resistance values.

8.12.2.2 Minimum Pile Spacing

Center-to-center pile spacing should not be less than the greater of 30 IN or 2.5 pile diameters or widths. A center-to-center spacing of less than 2.5 pile diameters may be considered on a case-by-case basis, subject to the approval of the WSDOT State Geotechnical Engineer and Bridge Design Engineer.

8.12.2.3 Piles Through Embankment Fill

Piles to be driven through embankments shall penetrate a minimum of 10 FT through original ground unless refusal on bedrock or competent bearing strata occurs at a lesser penetration. Fill used for embankment construction should be a select material, which does not obstruct pile penetration to the required depth. The maximum size of any rock particles in the fill should not exceed 6 IN.

Pre-drilling or spudding pile locations should be considered in situations where obstructions in the embankment fill cannot be avoided, particularly for displacement piles. Note that predrilling or spudding may reduce the pile skin friction and lateral resistance, depending on how the predrilling or spudding is conducted. The diameter of the predrilled or spudded hole, and the potential for caving of the hole before the pile is installed will need to be considered to assess the effect this will have on skin friction and lateral resistance.

8.12.2.4 Nearby Structures

Where pile foundations are placed adjacent to existing structures, the influence of the existing structure on the behavior of the foundation, and the effect of the foundation on the existing structures, including vibration effects due to pile installation, shall be investigated.

Vibration due to pile driving can cause settlement of existing foundations as well as structural damage to the adjacent facility. The combination of taking measures to mitigate the vibration levels through use of nondisplacement piles, predrilling, etc., and a good vibration monitoring program should be considered.

8.12.2.5 Determination of Pile Lateral Resistance

Pile foundations are subjected to horizontal loads due to wind, traffic loads, bridge curvature, vessel or traffic impact and earthquake. The nominal resistance of pile foundations to horizontal loads shall be evaluated based on both soil/rock and structural properties, considering soil-structure interaction. Determination of the soil/rock parameters required as input for design using soil-structure interaction methodologies is presented in **WSDOT GDM Chapter 5**.

Methods of analysis that use manual computation were developed by **Broms (1964a & b)**. They are discussed in detail by **Hannigan et al. (1997)**. Deep foundation horizontal movement at the foundation design stage may be analyzed using computer applications that consider soil-structure interaction.

Reese (1984) developed analysis methods that model the horizontal soil resistance using P-y curves. This analysis has been well developed and software is available for analyzing single piles and pile groups (**Reese, 1986; Williams et al, 2003; and Hannigan et al., 1997**). The analysis may be performed on a representative single pile with the appropriate pile top boundary condition or on the entire pile group.

Note that P-y methods generally apply to foundation elements that have some ability to bend and deflect. For large diameter, relatively short foundation elements (e.g., drilled shafts), the foundation element rotates rather than bends, in which case strain wedge theory (**Norris, 1986; Ashour, et al., 1998**) is more applicable – see **WSDOT GDM Section 8.13.4.7**. However, strain wedge theory can be adapted to more slender foundation elements as well.

Lateral resistance of single piles may be determined by static load test. If a static lateral load test is to be performed, it shall follow the procedures specified in ASTM 3966. Information on the methods of analysis and interpretation of lateral load tests are presented in the Handbook on Design of Piles and Drilled Shafts Under Lateral Load (**Reese, 1984**) and Static Testing of Deep Foundations (**Kyfor, et al., 1992**).

The lateral load response of the piles shall be modified to account for group effects. For P-y curves, the P-multipliers in **Table 8-21** should be used to modify the curves. If the pile cap will always be embedded, the P-y horizontal resistance of the soil on the cap face may be included in the horizontal resistance. Since many piles are installed in groups, the horizontal resistance of the group has been studied, and it has been found that multiple rows of piles will have less resistance than the sum of the single pile resistance. The front piles "shade" rows that are further back.

Horizontal load tests have been performed on pile groups, and multipliers (that are less than 1.0) have been determined that can be used in the analysis for the various rows. Those multipliers have been found to depend on the pile spacing and the row number in the direction of loading. Values from recent research have been tabulated by **Hannigan et al. (1997)**. Averaged values are provided in **Table 8-21**. To establish values of P_m for other pile spacing values, interpolation between values should be conducted.

Pile Center-to-Center spacing (in the direction of loading	Pile Load Modifiers, P_m		
	Row 1	Row 2	Row 3 and higher
3D	0.7	0.5	0.35
5D	1.0	0.85	0.7

Table 8-21 Pile Load Modifiers, P_m, for Multiple Row Shading (averaged from Hannigan, et al., 1997).

When the P-y method of analysis is used, the values of P shall be multiplied by the values, P_m, in **Table 8-21** to modify the P-y curves used in the analysis. The multipliers, P_m, in **Table 8-21** are a function of the center-to-center spacing of piles in the group in the direction of loading expressed in multiples of the pile diameter, D. The values of P_m in **Table 8-21** were developed for vertical piles only. Note that P_m is not applicable if strain wedge theory is used (see **WSDOT GDM Section 8.13.4.7**).

Loading direction and spacing are as defined in **Figure 8-27**. Note that if the loading direction for a single row of piles is perpendicular to the row (bottom right detail in the figure), a group reduction factor of less than 1.0 should only be used if the pile spacing is 5D or less (i.e., a P_m of 0.7 for a spacing of 3B), as shown in the detail.

Empirical data for pile spacings less than 3 pile diameters is very limited. If, due to space limitations, a smaller center-to-center spacing is used, subject to the requirements in **WSDOT GDM Section 8.12.2.2**, based on extrapolation of the values of P_m in **Table 8-21**, the following values of P_m at a spacing of no less than 2D may be used:

- For Row 1, $P_m = 0.45$
- For Row 2, $P_m = 0.33$
- For Row 3, $P_m = 0.25$

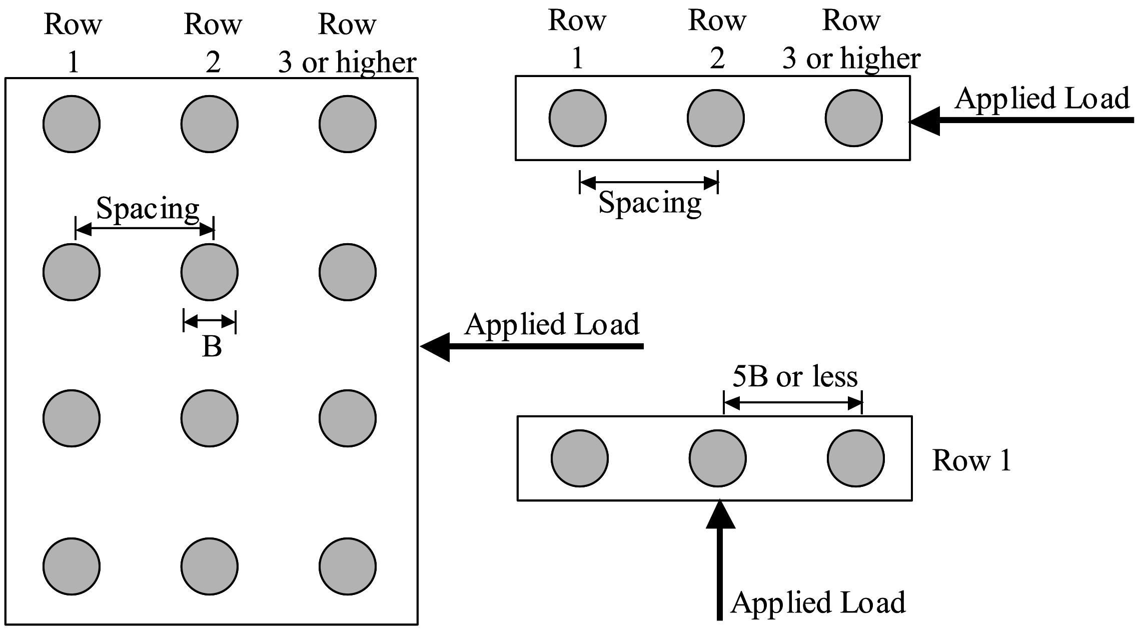


Figure 8-27 Definition of loading direction and spacing for group effects.

8.12.2.6 Batter Piles

When the lateral resistance of the soil surrounding the piles is inadequate to counteract the horizontal forces transmitted to the foundation, or when increased rigidity of the entire structure is required, batter piles may be considered for use in the foundation. WSDOT design preference is to avoid the use of batter piles unless no other structural option is available. Where negative skin friction loads are expected, batter piles should not be used. If batter piles are used in areas of significant seismic loading, the design of the pile foundation shall recognize the increased foundation stiffness that results.

In some cases, it may be desirable to use batter piles. From a general viewpoint, batter piles provide a much stiffer resistance to horizontal loads than would be possible with vertical piles. They can be very effective in resisting static horizontal loads. However, due to increased foundation stiffness, batter piles may not be desirable in resisting horizontal loads if the structure is located in an area where seismic loads are potentially high.

8.12.3 Service Limit State Design of Pile Foundations

Driven pile foundations shall be designed at the service limit state to meet the tolerable movements for the structure being supported in accordance with **WSDOT GDM Section 8.6.5.1**.

Service limit state design of driven pile foundations includes the evaluation of settlement due to static loads, and downdrag loads if present, overall stability, lateral squeeze, and lateral deformation. Overall stability of a pile supported foundation shall be evaluated where:

- The foundation is placed through an embankment,
- The pile foundation is located on, near or within a slope,
- The possibility of loss of foundation support through erosion or scour exists, or
- Bearing strata are significantly inclined.

In general, it is not desirable to subject the pile foundation to unbalance lateral loading caused by lack of overall stability or caused by lateral squeeze. The unbalanced lateral forces should be mitigated through stabilization measures, if possible.

Lateral analysis of pile foundations is conducted to establish the load distribution between the superstructure and foundations for all limit states, and to estimate the deformation in the foundation that will occur due to those loads. This section only addresses the evaluation of the lateral deformation of the foundation resulting from the distributed loads.

8.12.3.1 Settlement

For purposes of calculating the settlements of pile groups, loads shall be assumed to act on an equivalent footing based on the depth of embedment of the piles into the layer that provides support as shown in **figures 8-27 and 8-28**.

Pile group settlement shall be evaluated for pile foundations on cohesive soils. For piles tipped adequately into dense granular soils such that the equivalent footing is located on or within the dense granular soil, and furthermore are not subjected to downdrag loads, a detailed assessment of the pile group settlement may be waived. The load used in calculating pile group settlement shall be the permanently applied load on the foundation. Pile design should ensure that strength limit state considerations are satisfied before checking service limit state considerations.

For pile groups in clay or sand, pile group settlement shall be estimated using the equivalent footing location specified in **figures 8-27 and 8-28**, and the settlement estimating methodology for footings as specified in **WSDOT GDM Section 8.11.3.2**. In addition to the methods specified in **WSDOT GDM Section 8.11.3.2**, the settlement of pile groups in cohesionless soils may alternatively be taken as:

Using SPT:

$$\rho = \frac{qI\sqrt{B}}{N1_{60}} \tag{8-49}$$

Using CPT:

$$\rho = \frac{qBI}{2q_c} \tag{8-50}$$

in which:

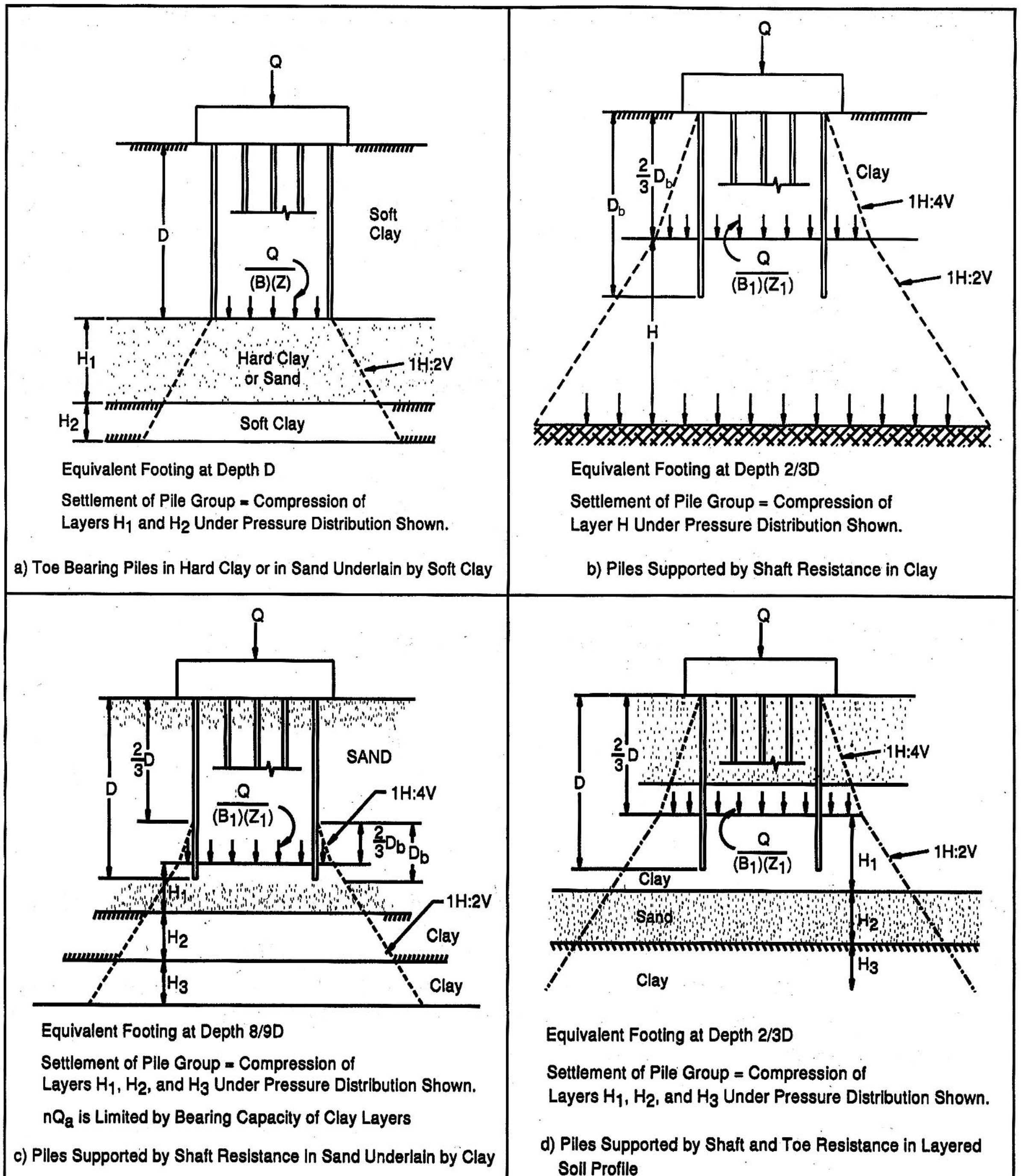
$$I = 1 - 0.125\frac{D'}{B} \geq 0.5 \qquad (8\text{-}51)$$

where:

ρ	=	settlement of pile group (IN)
q	=	net foundation pressure applied at $2D_b/3$, as shown in **Figure 8-28**; this pressure is equal to the applied load at the top of the group divided by the area of the equivalent footing and does not include the weight of the piles or the soil between the piles (KSF)
B	=	width or smallest dimension of pile group (FT)
I	=	influence factor of the effective group embedment (DIM)
D'	=	effective depth taken as $2D_b/3$ (FT)
D_b	=	depth of embedment of piles in layer that provides support, as specified in **Figure 8-28** (FT)
$N1_{60}$	=	SPT blow count corrected for both overburden and hammer efficiency effects (Blows/FT) as specified in **WSDOT GDM Chapter 5**.
q_c	=	static cone tip resistance (KSF)

The corrected SPT blow count or the static cone tip resistance should be averaged over a depth B below the equivalent footing. The SPT and CPT methods shall only be considered applicable to the distributions shown in **Figure 8-28b** and **Figure 8-29**.

This methodology based upon the use of empirical correlations proposed by **Meyerhof (1976)**. These are empirical correlations and the units of measure must match those specified for correct computations. This method may tend to over-predict settlements.



Notes:

(1) Plan area of perimeter of pile group = (B)(Z).

(2) Plan area $(B_1)(Z_1)$ = projection of area (B)(Z) at depth based on shown pressure distribution.

(3) For relatively rigid pile cap, pressure distribution is assumed to vary with depth as above.

(4) For flexible slab or group of small separate caps, compute pressures by elastic solutions.

Figure 8-28 Stress Distribution Below Equivalent Footing for Pile Group after Hannigan et al., (1997).

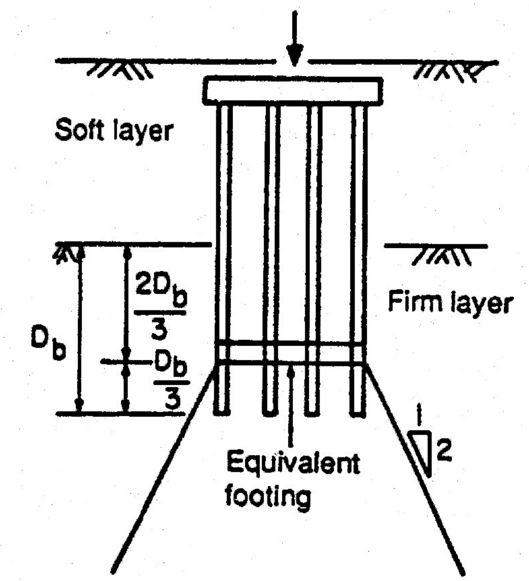


Figure 8-29 Location of Equivalent Footing (after Duncan and Buchignani 1976).

8.12.3.2 Overall Stability

The provisions of **WSDOT GDM Section 8.6.5.2** shall apply.

8.12.3.3 Horizontal Pile Foundation Movement

The horizontal movement of pile foundations shall be estimated using procedures that consider soil-structure interaction as specified in **WSDOT GDM Section 8.12.2.5**. Tolerable lateral movements of piles shall be established on the basis of confirming compatible movements of structural components (e.g., pile to column connections) for the loading condition under consideration.

The effects of the lateral resistance provided by an embedded cap may be considered in the evaluation of horizontal movement.

The orientation of non-symmetrical pile cross-sections shall be taken into account when computing the pile lateral stiffness.

The effects of group interaction shall be taken into account when evaluating pile group horizontal movement.

8.12.3.4 Settlement Due to Downdrag

The nominal pile resistance available to support structure loads plus downdrag shall be estimated by considering only the positive skin and tip resistance below the lowest layer contributing to the downdrag. In general, the available factored geotechnical resistance should be greater than the factored loads applied to the pile, including the downdrag, at the service limit state. In the instance where it is not possible to obtain adequate geotechnical resistance below the lowest layer contributing to downdrag (e.g., friction piles) to fully resist the downdrag, the structure should be designed to tolerate the full amount of settlement resulting from the downdrag and the other applied loads.

If adequate geotechnical resistance is available to resist the downdrag plus structure loads in the service limit state, the amount of deformation needed to fully mobilize the geotechnical resistance should be estimated, and the structure designed to tolerate the anticipated movement.

The static analysis procedures in **WSDOT GDM Section 8.12.4.7.5** may be used to estimate the available pile resistance to withstand the downdrag plus structure loads. Resistance may also be estimated using a dynamic method per **WSDOT GDM Section 8.12.4.7.2**, provided the skin friction resistance within the zone contributing to downdrag is subtracted from the resistance determined from the dynamic method during pile installation. The skin friction resistance within the zone contributing to downdrag may be estimated using the static analysis methods specified in **WSDOT GDM Section 8.12.4.7.5**, from signal matching analysis, or from pile load test results. Note that the static analysis methods may have bias that, on average, over or under predicts the skin friction. The bias of the method selected to estimate the skin friction should be taken into account as described in **WSDOT GDM Section 8.12.4.2**.

8.12.3.5 Lateral Squeeze

Bridge abutments supported on pile foundations driven through soft soils that are subject to unbalanced embankment fill loading shall be evaluated for lateral squeeze. Guidance on evaluating the potential for lateral squeeze and potential mitigation methods are included in **Hannigan et al., (1997)**.

8.12.4 Strength Limit State Geotechnical Design of Pile Foundations

For strength limit state design, the following shall be determined:

- Loads and performance requirements;
- Pile type, dimensions, and nominal axial pile resistance in compression;
- Size and configuration of the pile group to provide adequate foundation support;
- Estimated pile length to be used in the construction contract documents to provide a basis for bidding;
- A minimum pile penetration, if required, for the particular site conditions and loading, determined based on the maximum (deepest) depth needed to meet all of the applicable requirements identified in **WSDOT GDM Section 8.12.6**.
- The maximum driving resistance expected in order to reach the minimum pile penetration required (if applicable), including any soil/pile skin friction that will not contribute to the long-term nominal axial resistance of the pile (e.g., soil contributing to downdrag, or soil that will be scoured away);
- The drivability of the selected pile to achieve the required nominal axial resistance or minimum penetration with acceptable driving stresses at a satisfactory blow count per unit length of penetration; and
- The nominal structural resistance of the pile and/or pile group

A minimum pile penetration should only be specified if needed to insure that uplift, lateral stability, depth to resist downdrag, depth to resist scour, and depth for structural lateral resistance are met for the strength limit state, in addition to similar requirements for the service and extreme event limit states. See **WSDOT GDM Section 8.12.6** for additional details. Assuming dynamic methods (e.g., wave equation calibrated to dynamic measurements with signal matching analysis, pile formulae, etc.) are used during pile installation to establish when the bearing resistance has been met, a minimum pile penetration should not be used to insure that the required nominal pile bearing (i.e., compression) resistance is obtained.

A driving resistance exceeding the nominal bearing (compression) resistance required by the contract may be needed in order to reach a minimum penetration elevation specified in the contract.

The drivability analysis is performed to establish whether a hammer and driving system will likely install the pile in a satisfactory manner.

8.12.4.1 Point Bearing Piles on Rock

As applied to pile compressive resistance, this section shall be considered applicable to soft rock, hard rock, and very strong soils such as very dense glacial tills that will provide high nominal axial resistance in compression with little penetration.

If pile penetration into rock is expected to be minimal, the prediction of the required pile length will usually be based on the depth to rock.

A definition of hard rock that relates to measurable rock characteristics has not been widely accepted. Local or regional experience with driving piles to rock provides the most reliable definition.

In general, it is not practical to drive piles into rock to obtain significant uplift or lateral resistance. If significant lateral or uplift foundation resistance is required, drilled shaft foundations should be considered. If it is still desired to use piles, a pile drivability study should be performed to verify the feasibility of obtaining the desired penetration into rock.

8.12.4.1.1 Piles Driven to Soft Rock

Soft rock that can be penetrated by pile driving shall be treated in the same manner as soil for the purpose of design for axial resistance, in accordance with **WSDOT GDM Section 8.12.4.7**.

8.12.4.1.2 Piles Driven to Hard Rock

The nominal resistance of piles driven to point bearing on hard rock where pile penetration into the rock formation is minimal is controlled by the structural limit state. The nominal axial resistance shall not exceed the values obtained from Article 6.9.4.1 in the AASHTO LRFD Bridge Design Specifications with the resistance factors specified in Article 6.5.4.2 and Article 6.15 for severe driving conditions. A pile-driving acceptance criteria shall be developed that will prevent pile damage. Pile dynamic measurements should be used to monitor for pile damage when nominal axial resistances exceed 600 KIPS.

Care should be exercised in driving piles to hard rock to avoid tip damage. The tips of steel piles driven to hard rock should be protected by high strength, cast steel tip protection. If the rock is reasonably flat, the installation with pile tip protection will usually be successful. In the case of sloping rock, greater difficulty can arise and the use of tip protection with teeth should be considered. The geotechnical designer should also consider the following to minimize the risk of pile damage during installation:

- Use of a relatively small hammer. If a hydraulic hammer is used, it can be operated with a small stroke to seat the pile and then the axial resistance can be proven with a few larger hammer blows.
- If a larger hammer is used, specify a limited number of hammer blows after the pile tip reaches the rock. An example of a limiting criteria is five blows per one half inch.
- Extensive dynamic testing can be used to verify axial resistance on a large percentage of the piles. This approach could be used to justify larger design nominal resistances.

8.12.4.2 Prediction of Pile Length for the Contract-Required Nominal Axial Resistance

Subsurface geotechnical information combined with static analysis methods (**WSDOT GDM Section 8.12.4.7.5**), pre-construction test pile programs (**WSDOT GDM Section 8.12.9**), and/or pile load tests (**WSDOT GDM Section 8.12.4.7.1**) shall be used to estimate the depth of penetration required to achieve the desired nominal bearing for establishment of contract pile quantities. Local experience shall also be considered when making pile quantity estimates, both to select an estimation method and to assess the potential prediction bias for the method used (i.e., does the method selected tend to over-predict or under-predict pile compressive resistance?). If the depth of penetration required to obtain the desired nominal bearing (i.e., compressive) resistance is less than the depth required to meet the provisions of **WSDOT GDM Section 8.12.6**, the minimum penetration required per **WSDOT GDM Section 8.12.6** should be used as the basis for estimating contract pile quantities.

The estimated pile length required to support the required nominal resistance is determined using a static analysis; knowledge of the site subsurface conditions, and/or results from a pile load test. The pile length used to estimate quantities for the contract should also consider requirements to satisfy other design considerations, including service and extreme event limit states, as well as minimum pile penetration requirements for lateral stability, uplift, downdrag, scour, group settlement, etc.

One solution to the problem of predicting pile length is the use of a preliminary test program at the site. Such a program can range from a very simple operation of driving a few piles to evaluate drivability, to an extensive program where different pile types are driven and static and dynamic testing is performed.

In lieu of local experience, if a static analysis method is used to estimate the pile length required to achieve the desired nominal bearing for establishment of contract pile quantities, the factored resistance used to determine the size of the pile group required should be equated to the factored resistance estimated using the static analysis method as follows:

$$\varphi_{dyn} \text{ x } R_n = \varphi_{stat} \text{ x } R_{nstat} \qquad (8\text{-}52)$$

where,

φ_{dyn} =	the resistance factor for the dynamic method used to verify pile bearing resistance during driving (**Table 8-8**),
R_n =	the nominal pile bearing resistance (KIPS),
φ_{stat} =	the resistance factor for the static analysis method used to estimate the pile penetration depth required to achieve the desired bearing resistance (**Table 8-8**), and
R_{nstat} =	the predicted nominal resistance from the static analysis method used to estimate the penetration depth required (KIPS).

Using **Equation 8-52** and solving for R_{nstat}, use the static analysis method to determine the penetration depth required to obtain R_{nstat}.

The resistance factor for the static analysis method inherently accounts for the bias and uncertainty in the static analysis method. However, local experience may dictate that the penetration depth estimated using this approach be adjusted to reflect that experience.

Note that R_n is considered to be nominal bearing resistance of the pile needed to resist the applied loads, and is used as the basis for determining the resistance to be achieved during pile driving, R_{ndr} (see **WSDOT GDM Section 8.12.6 and 8.12.7**). R_{nstat} is only used in the static analysis method to estimate the pile penetration depth required.

8.12.4.3 Nominal Axial Resistance Change after Pile Driving

The potential for change in the nominal axial pile resistance after the end of pile driving should be evaluated. The effect of soil relaxation or setup should be included in the determination of nominal axial pile resistance for soils that are likely to be subject to these phenomena. Relaxation is not a common phenomenon but more serious than setup since it represents a reduction in the safety of the foundation. Pile setup can provide the opportunity for using larger pile nominal resistances at no increase in cost. However, it is necessary that the resistance gain be adequately proven. This is usually accomplished by re-strike testing with dynamic measurements. (**Komurka, et. al, 2003**).

8.12.4.3.1 Relaxation

If relaxation is possible in the soils at the site, the pile shall be tested in re-strike after a sufficient time has elapsed for relaxation to develop. Relaxation is a reduction in axial pile resistance. While relaxation typically occurs at the pile tip, it can also occur along the sides of the pile (**Morgano and White, 2004**). It can occur in dense sands or sandy silts and in some shales. Relaxation in the sands and silts will usually develop fairly quickly after the end of driving, perhaps in only a few minutes, as a result of the increase in the reduced pore pressure induced by dilation of the dense sands during driving. In some shales, relaxation occurs during the driving of adjacent piles and that will be immediate. There are other shales where the pile penetrates the shale and relaxation requires perhaps as much as two weeks to develop. In some cases, the amount of relaxation can be quite large. Since relaxation reduces nominal axial resistance, re-strike testing shall always be performed if relaxation is possible.

8.12.4.3.2 Setup

Setup in the nominal axial resistance may be used to support the applied load. Setup shall be proven after a specified length of time by re-striking the pile.

Setup is an increase in the nominal axial resistance that develops over time predominately along the pile shaft. Pore pressures increase during pile driving due to a reduction of the soil volume, reducing the effective stress and the shear strength. Setup may occur rapidly in cohesionless soils and more slowly in finer grained soils as excess pore water pressures dissipate. In some clays, setup may continue to develop over a period of weeks and even months, and in large pile groups it can develop even more slowly.

Setup (sometimes called pile "freeze") can be used to carry applied load, providing the opportunity for using larger pile nominal axial resistances, if it can be proven. Signal matching analysis of dynamic pile measurements made at the end of driving and later in re-strike can be an effective tool in evaluating and quantifying setup. (**Komurka, et al., 2003, Bogard & Matlock, 1990**).

Dynamic formula should in general not be used to evaluate pile axial resistance on re-strike, as these formulae were generally calibrated to end of drive data and inherently include some degree of setup. Higher degrees of confidence for evaluation of re-strike data are provided by pile dynamic measurements with signal matching analyses or static load tests.

Setup as it relates to the WSDOT dynamic formula is discussed further in **WSDOT GDM Section 8.12.4.7.4** and **Allen (2005b)**.

8.12.4.4 Buoyancy

Nominal axial resistance shall be determined using the groundwater level consistent with that used to calculate the effective stress along the pile sides and tip. The effect of hydrostatic pressure shall be considered in the design.

Unless the pile is bearing on rock, the tip resistance is primarily dependent on the effective surcharge that is directly influenced by the groundwater level. For drained loading conditions, the vertical effective stress is related to the ground water level and thus it affects pile axial resistance. Lateral resistance may also be affected.

Buoyant forces may also act on the pile if it is hollow and sealed so that water does not enter the pile. During pile installation, this may affect the driving resistance observed, especially in very soft soils.

8.12.4.5 Scour

The effect of scour, where scour can occur, shall be evaluated in selecting the pile penetration. The pile foundation shall be designed so that the pile penetration after the design scour event satisfies the required nominal axial and lateral resistance. The pile foundation shall be designed to resist debris loads occurring during the flood event in addition to the loads applied from the structure.

The resistance factors will be those used in the design without scour. The axial resistance of the material lost due to scour should be determined using a static analysis. The axial resistance of the material lost due to scour should not be factored, but consideration should be given to the bias of the static analysis method used to predict resistance. Method bias is discussed in **WSDOT GDM Section 8.12.4.2**.

The piles will need to be driven to the required nominal axial resistance plus the side resistance that will be lost due to scour. The resistance of the remaining soil is determined through field verification. The pile is driven to the required nominal axial resistance plus the magnitude of the skin friction lost as a result of scour, considering the prediction method bias.

Another approach that may be used takes advantage of dynamic measurements. In this case, the scour depth is determined, and the static analysis method is used to determine an estimated length. During the driving of test piles, the skin friction component of the axial resistance of the pile in the scourable material may be determined by a signal matching analysis of the dynamic measurements obtained when the pile is tipped below the scour elevation. The material below the scour elevation must provide the required nominal resistance after scour occurs.

If a static analysis method is used to determine the final pile bearing resistance (i.e., a dynamic analysis method is not used to verify pile resistance as driven), the available bearing resistance, and the pile tip penetration required to achieve the desired bearing resistance, shall be determined assuming that the soil subject to scour is completely removed, resulting in no overburden stress at the bottom of the scour zone.

In some cases, the flooding stream will carry debris that will induce horizontal loads on the piles.

Additional information regarding pile design for scour is provided in **Hannigan, et al., (1997)**.

Pile design for scour is illustrated in **Figure 8-29**, where,

R_{scour} = skin friction which must be overcome during driving through scour zone (KIPS)
Q_p = $(\Sigma\gamma_i Q_i)$ = factored load per pile (KIPS)
$D_{est.}$ = estimated pile length needed to obtain desired nominal resistance per pile (FT)
φ_{dyn} = resistance factor, assuming that a dynamic method is used to estimate pile resistance during installation of the pile (if a static analysis method is used instead, use φ_{stat})

From **Equation 8-1**, the summation of the factored loads $(\Sigma\gamma_i Q_i)$ must be less than or equal to the factored resistance (φR_n). Therefore, the nominal resistance R_n must be greater than or equal to the sum of the factored loads divided by the resistance factor φ. Hence, the nominal bearing resistance of the pile needed to resist the factored loads is therefore,

$$R_n = (\Sigma\gamma_i Q_i)/\varphi_{dyn} \qquad (8\text{-}53)$$

If dynamic pile measurements or dynamic pile formula are used to determine final pile bearing resistance during construction, the resistance that the piles are driven to must be adjusted to account for the presence of the soil in the scour zone. The total driving resistance, R_{ndr}, needed to obtain R_n, accounting for the skin friction that must be overcome during pile driving that does not contribute to the design resistance of the pile is as follows:

$$R_{ndr} = R_{scour} + R_n \tag{8-54}$$

Note that R_{scour} remains unfactored in this analysis to determine R_{ndr}.

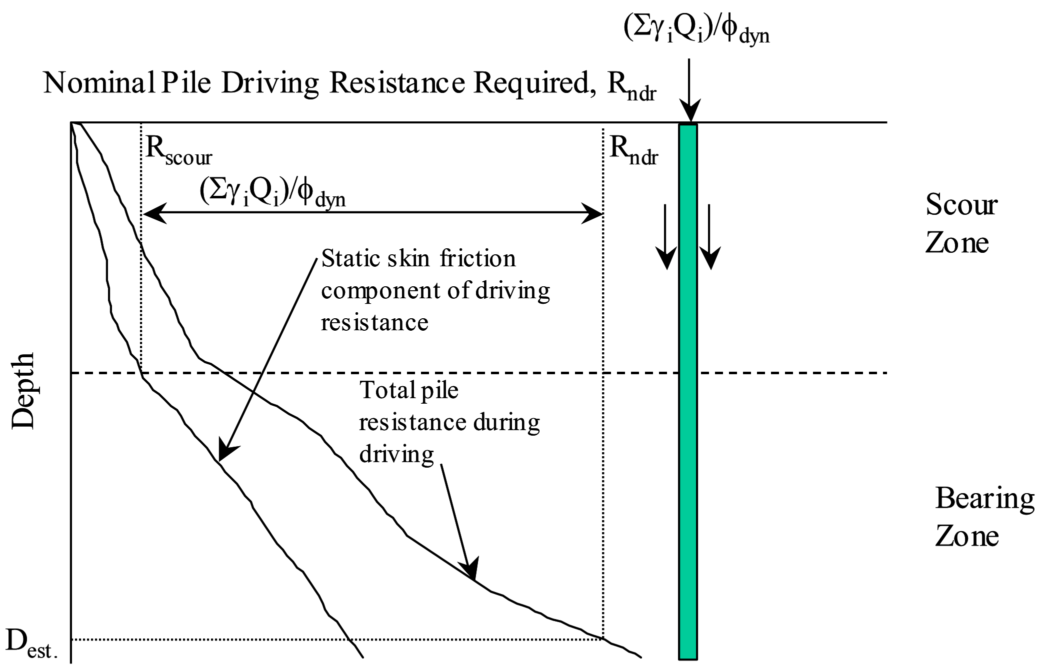


Figure 8-30 Design of pile foundations for scour.

8.12.4.6 Downdrag

The foundation should be designed so that the available factored geotechnical resistance is greater than the factored loads applied to the pile, including the downdrag, at the strength limit state. The nominal pile resistance available to support structure loads plus downdrag shall be estimated by considering only the positive skin and tip resistance below the lowest layer contributing to the downdrag. The pile foundation shall be designed to structurally resist the downdrag plus structure loads.

Pile design for downdrag is illustrated in **Figure 8-31**, where,

R_{Sdd} =	skin friction which must be overcome during driving through downdrag zone (KIPS)
Q_p =	$(\Sigma\gamma_i Q_i)$ = factored load per pile, excluding downdrag load (KIPS)
DD =	downdrag load per pile (KIPS)
$D_{est.}$ =	estimated pile length needed to obtain desired nominal resistance per pile (FT)
φ_{dyn} =	resistance factor, assuming that a dynamic method is used to estimate pile resistance during installation of the pile (if a static analysis method is used instead, use φ_{stat})
γ_p =	load factor for downdrag

Similar to the derivation of **Equation 8-53**, the nominal bearing resistance of the pile needed to resist the factored loads, including downdrag, is therefore,

$$R_n = (\Sigma\gamma_i Q_i)/\varphi_{dyn} + \gamma_p DD/\varphi_{dyn} \qquad (8\text{-}55)$$

The total nominal driving resistance, R_{ndr}, needed to obtain R_n, accounting for the skin friction that must be overcome during pile driving that does not contribute to the design resistance of the pile, is as follows:

$$R_{ndr} = R_{Sdd} + R_n \qquad (8\text{-}56)$$

where, R_{ndr} is the nominal pile driving resistance required. Note that R_{Sdd} remains unfactored in this analysis to determine R_{ndr}.

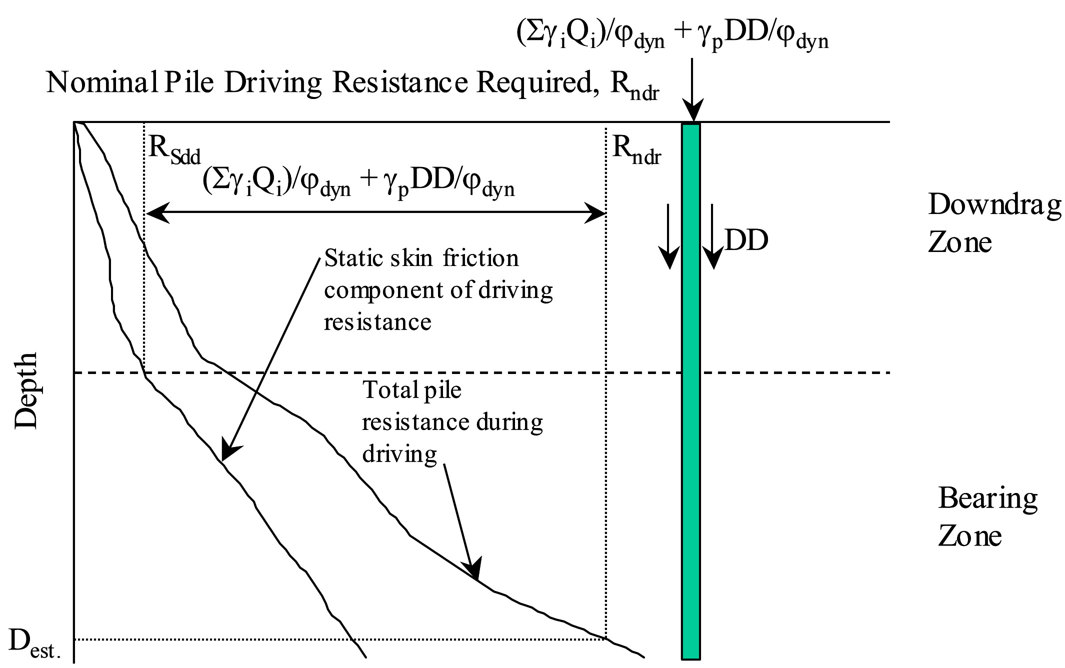


Figure 8-31 Design of pile foundations for downdrag.

In the instance where it is not possible to obtain adequate geotechnical resistance below the lowest layer contributing to downdrag (e.g., friction piles) to fully resist the downdrag, or if it is anticipated that significant deformation will be required to mobilize the geotechnical resistance needed to resist the factored loads including the downdrag load, the structure should be designed to tolerate the settlement resulting from the downdrag and the other applied loads in accordance with **WSDOT GDM Section 8.12.3.4**.

The static analysis procedures in **WSDOT GDM Section 8.12.4.7.5** may be used to estimate the available pile resistance to withstand the downdrag plus structure loads to estimate pile lengths required to achieve the required bearing resistance. For this calculation, it should be assumed that the soil subject to downdrag still contributes overburden stress to the soil below the downdrag zone.

Resistance may also be estimated using a dynamic method per **WSDOT GDM Section 8.12.4.7.2**, provided the skin friction resistance within the zone contributing to downdrag is subtracted from the resistance determined from the dynamic method during pile installation. The skin friction resistance within the zone contributing to downdrag may be estimated using the static analysis methods specified in **WSDOT GDM Section 8.12.4.7.5**, from signal matching analysis, or from pile load test results. Note that the static analysis method may have a bias, on average over or under predicting the skin friction. The bias of the method selected to estimate the skin friction within and above the downdrag zone should be taken into account as described in **WSDOT GDM Section 8.12.4.2**.

8.12.4.7 Determination of Nominal Axial Pile Resistance in Compression

Pile nominal axial resistance should be field verified during pile installation using load tests, dynamic tests**,** wave equation or dynamic formula. The resistance factor selected for design shall be based on the method used to verify pile axial resistance. The production piles shall be driven to the minimum blow count determined from the static load test, or dynamic test or formula used unless a deeper penetration is required due to uplift, scour or lateral resistance requirements, or other requirements as specified in **WSDOT GDM Section 8.12.6**.

If it is determined that dynamic methods are unsuitable for field verification of nominal axial resistance, and a static analysis method is used without verification of axial resistance during pile driving by static load test, dynamic test or formula, the piles shall be driven to the tip elevation determined from the static analysis, and to meet other limit states as required in **WSDOT GDM Section 8.12.6**.

This section addresses the determination of the nominal bearing (compression) resistance needed to meet strength limit state requirements, using factored loads and factored resistance values. From this design step, the number of piles and pile resistance needed to resist the factored loads applied to the foundation are determined. Both the loads and resistance values are factored for this determination.

8.12.4.7.1 Static Load Test

If a static pile load test is used to determine the pile axial resistance, the test shall not be performed less than 5 days after the test pile was driven unless approved by the geotechnical designer. The load test shall follow the procedures specified in ASTM D 1143, and the loading procedure should follow the Quick Load Test Method, unless detailed longer-term load-settlement data is needed, in which case the standard loading procedure should be used. The pile axial resistance shall be determined from the test data using:

- The Davisson Method for piles 24 inches or less in diameter (length of side for square piles),
- At a pile top movement, s_f (IN), as determined from **Equation 8-57** for piles larger than 36 inches in diameter (length of side for square piles), and
- For piles greater than 24 inches but less than 36 inches in diameter, a criterion to determine the pile axial resistance that is linearly interpolated between the criteria determined at diameters of 24 and 36 inches.

unless specified otherwise by the geotechnical designer.

$$s_f = \frac{QL}{AE} + \frac{B}{30} \qquad (8\text{-}57)$$

where:

Q = test load (KIPS)

L = pile length (IN)

A = pile cross sectional area (IN^2)

E = pile modulus (KSI)

B = pile diameter (length of side for square piles) (IN)

B = pile diameter (length of side for square piles) (IN)

The Quick Test Procedure is desirable because it avoids problems that frequently arise when performing a static test that cannot be started and completed within an eight-hour period. Tests that extend over a longer period are difficult to perform due to the limited number of experienced personnel that are usually available. The Quick Test has proven to be easily performed in the field and the results usually are satisfactory. However, if the formation in which the pile is installed may be subject to significant creep settlement, alternative procedures provided in ASTM D1143 should be considered.

The Davisson Method of axial resistance evaluation is performed by constructing a line on the load test curve that is parallel to the elastic compression line of the pile. The elastic compression line is calculated by assuming equal compressive forces are applied to the pile ends. The elastic compression line is offset by a specified amount of displacement. The Davisson Method is illustrated in **Figure 8-32** and described in more detail in **Hannigan, et al., (1997)**. For piles with large cross sections, the Davisson Method will under predict the pile nominal axial resistance.

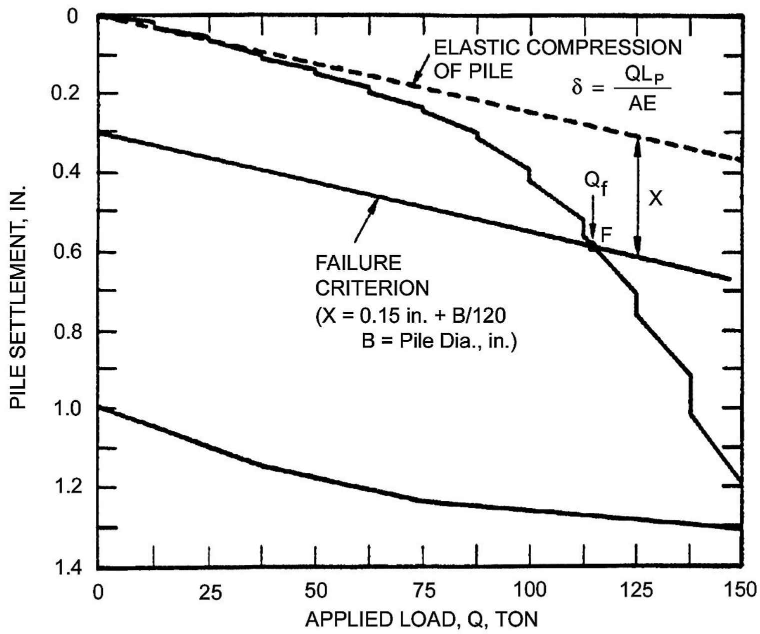


Figure 8-32 Alternate Method Load Test Interpretation (Cheney & Chassie, 2000, modified after Davisson, 1972).

Driving criteria should be established from the pile load test results using one of the following approaches:

1. Use dynamic measurements with signal matching analysis calibrated to match the pile load test results; a dynamic test shall be performed on the static test pile at the end of driving and again as soon as possible after completion of the static load test by re-strike testing. The signal matching analysis of the re-strike dynamic test should then be used to produce a calibrated signal matching analysis that matches the static load test result. Perform additional production pile dynamic tests with calibrated signal matching analysis (see **Table 8-10** for the number of tests required) to develop the final driving criteria.
2. If dynamic test results are not available, use the pile load test results to calibrate a wave equation analysis, matching the wave equation prediction to the measured pile load test resistance, in consideration of the hammer used to install the load test pile.
3. For the case where the bearing stratum is well defined, relatively uniform in extent, and consistent in its strength, driving criteria may be developed directly from the pile load test result(s), and should include a minimum driving resistance combined with a minimum hammer delivered energy to obtain the required bearing resistance. In this case, the hammer used to drive the pile(s) that are load tested shall be used to drive the production piles.
4. For the case where driving to a specified tip elevation without field verification using dynamic methods is acceptable and dynamic methods are determined to be unsuitable for field verification of nominal axial resistance (see **WSDOT GDM Section 8.9**), the load test results may be used to calibrate a static pile resistance analysis method (see **WSDOT GDM Section 8.12.4.7.1**). The calibrated static analysis method should then be used to determine the depth of penetration into the bearing zone needed to obtain the desired nominal pile resistance. In this case, the bearing zone shall be well defined based on subsurface test hole or probe data.

The specific application of the four driving criteria development approaches provided herein may be site specific, and may also depend on the degree of scatter in the pile load test and dynamic test results. If multiple load tests and dynamic tests with signal matching are conducted at a given site as defined in **WSDOT GDM Section 8.9**, the geotechnical designer will need to decide how to "average" the results to establish the final driving criteria for the site, and if local experience is available, in consideration of that local experience. Furthermore, if one or more of the pile load tests yield significantly higher or lower nominal resistance values than the other load tests at a given project site, the reason for the differences should be thoroughly investigated before simply averaging the results together or treating the result(s) as anomalous.

Regarding the first driving criteria development approach provided herein, the combination of the pile load and dynamic test results should be used to calibrate a wave equation analysis to apply the test results to production piles not subjected to dynamic testing, unless all piles are dynamically tested. For piles not dynamically tested, hammer performance should still be assessed to insure proper application of the driving criteria. Hammer performance assessment should include stroke measurement for hammers that have a variable stroke, bounce chamber pressure measurement for double acting hammers, or ram velocity measurement for hammers that have a fixed stroke. Hammer performance assessment should also be conducted for the second and third driving criteria development approaches.

Regarding the fourth driving criteria development approach provided herein, it is very important to have the bearing zone well defined at each specific location within the site where piles are to be driven. Additional test borings beyond the minimums specified in **Table 8-2** will likely be necessary to obtain an adequately reliable foundation when using this driving criteria development approach. Note that a specific resistance factor for this approach to using load test data to establish the driving criteria is not provided. While some improvement in the reliability of the static analysis method calibrated for the site in this manner is likely, no statistical data are currently available from which to fully assess reliability and establish a resistance factor. Therefore, the resistance factor for the static analysis method used should be used for the pile foundation design.

Note that it may not be possible to calibrate the dynamic measurements with signal matching analysis to the pile load test results if the driving resistance at the time the dynamic measurement is taken is too high (i.e., the pile set per hammer blow is too small). In this case, adequate hammer energy is not reaching the pile tip to assess end bearing and produce an accurate match, though in such cases, the prediction will usually be quite conservative. In general, a tip movement (pile set) of 0.10 to 0.15 inch is needed to provide an accurate signal matching analysis.

In cases where a significant amount of soil setup occurs, a more accurate result may be obtained by combining the end bearing determined using the signal matching analysis obtained for the end of driving (EOD) with the signal matching analysis for the side friction at the beginning of redrive (BOR).

8.12.4.7.2 Dynamic Testing

Dynamic testing shall be performed according to the procedures given in ASTM D 4945. If possible, the dynamic test should be performed as a re-strike test if the geotechnical designer anticipates significant time dependent strength change. The pile nominal axial resistance shall be determined by a signal matching analysis of the dynamic pile test data if the dynamic test is used to establish the driving criteria.

Additional dynamic testing may be used for quality control during the driving of production piles. In this case, the dynamic test shall be calibrated as specified in **WSDOT GDM Section 8.12.4.7.1** by the results of the static load test or signal matching analysis used to establish the nominal axial resistance, in combination with the Case Method (**Rausche et al. 1985)**.

If additional dynamic testing is used for pile bearing resistance quality control, pile bearing resistance should be determined using the Case Method analysis as described by **Rausche et al. (1985)**.

If the Case method is used to estimate pile bearing resistance where a pile load test is not performed, the damping constant j in the Case Method shall be selected (i.e., calibrated) so it gives the axial resistance obtained by a signal matching analysis. When static load tests for the site as defined in **WSDOT GDM Section 8.9.2** have been performed, the damping constant j in the Case Method shall be selected (i.e., calibrated) so it gives the axial resistance obtained by the static load test.

Driving criteria should be developed using the results of dynamic tests with signal matching analysis to calibrate a wave equation analysis, matching the wave equation prediction to the resistance predicted from the signal matching analysis, to extrapolate the dynamic test/signal matching results to piles not dynamically tested. If all piles are dynamically tested, the resistance predicted from the dynamic test using the Case Method, using "j" calibrated to match the signal matching results should be used to verify pile production resistance.

The dynamic test may be used to establish the driving criteria at the beginning of production driving. The minimum number of piles that should be tested are as specified in **Table 8-10**. A signal matching analysis (**Rausche, et al., 1972**) of the dynamic test data should always be used to determine axial resistance if a static load test is not performed. See **Hannigan, et al. (1997)** for a description of and procedures to conduct a signal matching analysis. Re-strike testing should be performed if setup or relaxation is anticipated.

Dynamic testing and interpretation of the test data should only be performed by certified, experienced testers.

8.12.4.7.3 Wave Equation Analysis

A wave equation analysis may be used to establish the driving criteria. In this case, the wave equation analysis shall be performed based on the hammer and pile driving system to be used for pile installation. To avoid pile damage, driving stresses shall not exceed the values obtained in **WSDOT GDM Section 8.12.8**, using the resistance factors specified or referred to in **Table 8-8**. Furthermore, the blow count needed to obtain the maximum driving resistance anticipated shall be less than the maximum value established based on the provisions in **WSDOT GDM Section 8.12.8**.

A wave equation analysis should also be used to evaluate pile drivability.

Note that without dynamic test results with signal matching analysis and/or pile load test data (see **WSDOT GDM Section 8.12.4.7.2**), considerable judgment is required to use the wave equation to predict the pile bearing resistance. Key soil input values that affect the predicted resistance include the soil damping and quake values, the skin friction distribution (e.g., such as could be obtained from a pile bearing static analysis), and the anticipated amount of soil setup or relaxation. Furthermore, the actual hammer performance is a variable that can only be accurately assessed through dynamic measurements, though "standard" input values are available. The resistance factor of 0.40 provided in **Table 8-8** for the wave equation was developed from calibrations performed by **Paikowsky, et al. (2004)**, in which default wave equation hammer and soil input values were used. Therefore, their wave equation calibrations did not consider the potential improved pile resistance prediction reliability that could result from measurement of at least some of these key input values. It is for these reasons that the resistance factor specified in **WSDOT GDM Section 8.9** is relatively low (see **Paikowsky, et al., 2004**, for additional information regarding the development of the resistance factor for the wave equation). If additional local experience or site specific test results are available to allow the wave equation soil or hammer input values to be refined and made more accurate, a higher resistance factor may be used.

The wave equation can be used in combination with dynamic test results with signal matching analysis and/or pile load test data to provide the most accurate wave equation pile resistance prediction. Such data are used to calibrate the wave equation, allowing the resistance factor for dynamic testing and signal matching specified in **WSDOT GDM Section 8.9** to be used.

8.12.4.7.4 Dynamic Formula

If a dynamic formula is used to establish the driving criterion in lieu of a combination of dynamic measurements with signal matching per **WSDOT GDM Section 8.12.4.7.2**, wave equation analysis, and/or pile load tests, the WSDOT Pile Driving Formula from the WSDOT Standard Specifications for Roads, Bridge, and Municipal Construction Section 6-05.3(12) shall be used, unless otherwise specifically approved by the WSDOT State Geotechnical Engineer.

The hammer energy used to calculate the nominal (ultimate) pile resistance during driving in the WSDOT and other driving formulae described herein is the developed energy. The developed hammer energy is the actual amount of gross energy produced by the hammer for a given blow. This value will never exceed the rated hammer energy (rated hammer energy is the maximum gross energy the hammer is capable of producing, i.e., at its maximum stroke).

The development of the WSDOT pile driving formula is described in **Allen (2005b)**. The nominal (ultimate) pile resistance during driving using this method shall be taken as:

$$R_{ndr} = F \times E \times Ln\left(10N\right) \qquad (8\text{-}58)$$

Where:

R_{ndr}	=	driving resistance, in TONS
F	=	1.8 for air/steam hammers
	=	1.2 for open ended diesel hammers and precast concrete piles
	=	1.6 for open ended diesel hammers and steel or timber piles
	=	1.2 for closed ended diesel hammers
	=	1.9 for hydraulic hammers
	=	0.9 for drop hammers
E	=	developed energy, equal to W times H[1], in ft-kips
W	=	weight of ram, in kips

H	=	vertical drop of hammer or stroke of ram, in feet
N	=	average penetration resistance in blows per inch for the last 4 inches of driving
Ln	=	the natural logarithm, in base "e"

[1]For closed-end diesel hammers (double-acting), the developed hammer energy (E) is to be determined from the bounce chamber reading. Hammer manufacturer calibration data may be used to correlate bounce chamber pressure to developed hammer energy. For double acting hydraulic and air/steam hammers, the developed hammer energy shall be calculated from ram impact velocity measurements or other means approved by the Engineer. For open ended diesel hammers (single-acting), the blows per minute may be used to determine the developed energy (E).

Note that R_{ndr} as determined by this driving formula is presented in units of TONS rather than KIPS, to be consistent with the WSDOT Standard Specifications for Road, Bridge, and Municipal Construction (M 41-10). The above formula applies only when:

1. The hammer is in good condition and operating in a satisfactory manner;
2. A follower is not used;
3. The pile top is not damaged;
4. The pile head is free from broomed or crushed wood fiber;
5. The penetration occurs at a reasonably quick, uniform rate; and the pile has been driven at least 2 feet after any interruption in driving greater than 1 hour in length.
6. There is no perceptible bounce after the blow. If a significant bounce cannot be avoided, twice the height of the bounce shall be deducted from "H" to determine its true value in the formula.
7. For timber piles, bearing capacities calculated by the formula above shall be considered effective only when it is less than the crushing strength of the piles.
8. If "N" is greater than or equal to 1.0 blow/inch.

As described in detail in **Allen (2005b)**, **Equation 8-58** should not be used for nominal pile bearing resistances greater than approximately 1,200 KIPS (600 TONS), or for pile diameters greater than 30 inches, due to the paucity of data available to verify the accuracy of this equation at higher resistances and larger pile diameters, and due to the increased scatter in the data.

As is true of most driving formulae, if they have been calibrated to pile load test results, the WSDOT pile driving formula has been calibrated to N values obtained at end of driving (EOD). Since the pile nominal resistance obtained from pile load tests are typically obtained days, if not weeks, after the pile has been driven, the gain in pile resistance that typically occurs with time is in effect correlated to the EOD N value through the driving formula. That is, the driving formula assumes that an "average" amount of setup will occur after EOD when the pile nominal resistance is determined from the formula (see **Allen, 2005b**). Hence, the WSDOT driving formula shall not be used in combination with the resistance factor φ_{dyn} provided in **Table 8-8** for beginning of redrive (BOR) N values to obtain nominal resistance. If pile foundation nominal resistance must be determined based on restrike (BOR) driving resistance, dynamic measurements in combination with signal matching analysis and/or pile load test results should be used.

Since driving formulas inherently account for a moderate amount of pile resistance setup, it is expected that theoretical methodologies such as the wave equation will predict lower nominal bearing resistance values for the same driving resistance N than empirical methodologies such as the WSDOT driving formula. This should be considered when assessing pile drivability (see **WSDOT GDM Section 8.12.8**), if it is intended to evaluate the pile/hammer system for contract approval purposes using the wave equation, but using a pile driving formula for field determination of pile nominal bearing resistance.

If a dynamic (pile driving) formula other than the one provided here is used, subject to the approval of the State Geotechnical Engineer, it shall be calibrated based on measured load test results to obtain an appropriate resistance factor, consistent with **WSDOT GDM Section 8.9** and **Allen (2005b)**.

If a dynamic formula is used, the structural compression limit state cannot be treated separately as with the other axial resistance evaluation procedures unless a drivability analysis if performed. Evaluation of pile drivability, including the specific evaluation of driving stresses and the adequacy of the pile to resist those stresses without damage, is strongly recommended. When drivability is not checked, it is necessary that the pile design stresses be limited to values that will assure that the pile can be driven without damage. For steel piles, guidance is provided in Article 6.15.2 of the AASHTO LRFD Bridge Design Specifications for the case where risk of pile damage is relatively high. If pile drivability is not checked, it should be assumed that the risk of pile damage is relatively high. For concrete piles and timber piles, no specific guidance is available in Sections 5 and 8, respectively, of the AASHTO LRFD Bridge Design Specifications regarding safe design stresses to reduce the risk of pile damage. In past practice (see **AASHTO 2002**), the required nominal axial resistance has been limited to 0.6 f_c' for concrete piles and 2,000 psi for timber piles if pile drivability is not evaluated.

8.12.4.7.5 Static Analysis

When a static analysis prediction method is used to determine pile installation criteria (i.e., for bearing resistance), the nominal pile resistance shall be factored at the strength limit state using the resistance factors in **Table 8-8** associated with the method used to compute the nominal bearing resistance of the pile. The factored bearing resistance of piles, R_R, may be taken as:

$$R_R = \varphi_{stat} R_n \qquad (8\text{-}59)$$

or:

$$R_R = \varphi_{stat} R_n = \varphi_{stat} R_p + \varphi_{stat} R_s \qquad (8\text{-}60)$$

in which:

$$R_p = q_p A_p \qquad (8\text{-}61)$$

$$R_s = q_s A_s \qquad (8\text{-}62)$$

where:

φ_{stat}	=	resistance factor for the bearing resistance of a single pile specified in **Table 8-8**
R_p	=	pile tip resistance (KIPS)
R_s	=	pile side resistance (KIPS)
q_p	=	unit tip resistance of pile (KSF)
q_s	=	unit side resistance of pile (KSF)

A_s = surface area of pile side (FT^2)
A_p = area of pile tip (FT^2)

Both total stress and effective stress methods may be used, provided the appropriate soil strength parameters are available. The limitations of each method as described in **WSDOT GDM Section 8.9.2** should be applied in the use of these static analysis methods. The resistance factors for the skin friction and tip resistance, estimated using these methods, shall be as specified in **Table 8-8**. Note that if the pile tip is in a different material than the pile shaft, different resistance factors that are consistent with the design method used to estimate skin friction and tip resistance for pile may be needed for the side resistance and tip resistance calculations.

While the most common use of static analysis methods is solely for estimating pile quantities, a static analysis may be used to establish pile installation criteria if dynamic methods are determined to be unsuitable for field verification of nominal axial resistance. This is applicable on projects where pile quantities are relatively small, pile loads are relatively low, and/or where the setup time is long so that re-strike testing would require an impractical wait-period by the Contractor on the site (e.g., soft silts or clays where a large amount of setup is anticipated).

When a static analysis method is used to estimate pile nominal bearing resistance, the side and end bearing resistance of the piles should be determined using one of the methods described in the sections that follow, as applicable for the site conditions. All of the methods provided in the sections that follow shall be performed as described in each of the source references as listed, or as described in **Hannigan, et al. (1997)**.

8.12.4.7.5(a)α-Method

The α -method, based on total stress, may be used to relate the adhesion between the pile and a clay to the undrained strength of the clay. The method shall be performed as described by **Tomlinson (1980)**.

8.12.4.7.5(b)β -Method

The β-method, based on effective stress, may be used for predicting skin friction of prismatic piles. The method shall be performed as described by **Esrig and Kirby (*1979*)**. The β-method has been found to work best for piles in normally consolidated and lightly overconsolidated clays. The method tends to overestimate skin friction of piles in heavily overconsolidated soils. **Esrig and Kirby (*1979*)** suggested that for heavily overconsolidated clays, the value of β should not exceed 2.

8.12.4.7.5(c)λ-Method

The λ-method, based on effective stress (though it does contain a total stress parameter), may be used to relate the unit skin friction to passive earth pressure. The value of λ decreases with pile length and was found empirically by examining the results of load tests on steel pipe piles. The method shall be performed as described by **Vijayvergiya and Focht (1972)**.

8.12.4.7.5(d) Tip Resistance in Cohesive Soils

The nominal unit tip resistance of piles in saturated clay, in KSF, shall be taken as:

$$q_p = 9S_u \tag{8-63}$$

S_u = undrained shear strength of the clay near the pile base (KSF)

8.12.4.7.5(e) Nordlund/Thurman Method in Cohesionless Soils

This method was derived based on load test data for piles in sand (**Nordlund, 1963**). Therefore, this method should be applied only to sands and non-plastic silts. In practice, it has been used for gravelly soils as well. Detailed design procedures for the Nordlund/Thurman method are provided in **Nordlund (1979)** and **Hannigan, et al., (1997)**. For H-piles the perimeter, or "box" area should generally be used to compute the surface area of the pile side. When calculating tip resistance by this method, if the friction angle, ϕ, is estimated from average, corrected SPT blow counts, $N1_{60}$, the $N1_{60}$ values should be averaged over the zone from the pile tip to 2 diameters below the pile tip.

8.12.4.7.5(f) Using SPT or CPT in Cohesionless Soils

In-situ tests are widely used in cohesionless soils because obtaining good quality samples of cohesionless soils is very difficult. In-situ test parameters may be used to estimate the tip resistance and skin friction of piles. Two frequently used in-situ test methods for predicting pile axial resistance are the standard penetration test (SPT) method (**Meyerhof 1976**) and the cone penetration test (CPT) method (**Nottingham and Schmertmann 1975**). These methods should be applied only to sands and nonplastic silts.

For SPT data, the Method as described by **Meyerhof (1976)** may be used. SPT N values corrected for overburden pressure and SPT hammer efficiency ($N1_{60}$) as described in **WSDOT GDM Chapter 5** shall be used with this method.

The Meyerhof method provides procedures for both displacement and nondisplacement piles. Displacement piles, which have solid sections or hollow sections with a closed end, displace a relatively large volume of soil during penetration. Non-displacement piles usually have relatively small cross-sectional areas, e.g., steel H-piles and open-ended pipe piles that have not yet plugged. Plugging occurs when the soil between the flanges in a steel H-pile or the soil in the cylinder of an open-ended steel pipe pile adheres fully to the pile and moves down with the pile as it is driven.

For CPT data, the method as described by **Nottingham and Schmertmann (1975)** may be used.

CPT may be used to determine:

- The cone penetration resistance, q_c, which may be used to determine the tip resistance of piles, and
- Sleeve friction, f_s, which may be used to determine the skin friction resistance.

The minimum average cone resistance between 0.7 and 4 pile diameters below the elevation of the pile tip shall be obtained by a trial and error process, with the use of the minimum-path rule. The minimum-path rule shall also be used to find the value of cone resistance for the soil for a distance of eight pile diameters above the tip. The two results shall be averaged to determine the pile tip resistance. **Nottingham and Schmertmann (1975)** found that using a weighted average core resistance gives a good estimation of tip resistance in piles for all soil types.

8.12.4.8 Resistance of Pile Groups in Compression

For pile groups in clay, the nominal axial resistance of the pile group shall be taken as the lesser of:

- The sum of the individual nominal resistances of each pile in the group, or
- The nominal resistance of an equivalent pier consisting of the piles and the block of soil within the area bounded by the piles.

If the cap is not in firm contact with the ground and if the soil at the surface is soft, the individual resistance of each pile shall be multiplied by an efficiency factor η, taken as:

- $\eta = 0.65$ for a center-to-center spacing of 2.5 diameters,
- $\eta = 1.0$ for a center-to-center spacing of 6.0 diameters.
- For intermediate spacings, the value of η may be determined by linear interpolation.

If the cap is in firm contact with the ground, no reduction in efficiency shall be required. If the cap is not in firm contact with the ground and if the soil is stiff, no reduction in efficiency shall be required.

The bearing capacity of pile groups in cohesionless soil shall be the sum of the resistance of all the piles in the group. The efficiency factor, η, shall be 1.0 where the pile cap is or is not in contact with the ground for a center-to-center pile spacing of 2.5 diameters or greater. The resistance factor is the same as that for single piles, as specified in **Table 8-8.**

For pile groups in clay or sand, if a pile group is tipped in a strong soil deposit overlying a weak deposit, the block bearing resistance shall be evaluated with consideration to pile group punching as a group into the underlying weaker layer. The methods in **WSDOT GDM Section 8.11.4.1.1** of determining bearing resistance of a spread footing in a strong layer overlying a weaker layer shall apply, with the notional footing located as shown in **WSDOT GDM Section 8.12.3.1**.

The equivalent pier approach checks for block failure and is generally only applicable for pile groups within cohesive soils. For pile groups in sand, the sum of the nominal resistances of the individual piles always controls the group resistance.

When analyzing the equivalent pier, the full shear strength of the soil should be used to determine the friction resistance. The total base area of the equivalent pier should be used to determine the end bearing resistance.

In cohesive soils, the resistance of a pile group depends on whether the cap is in firm contact with the ground beneath. If the cap is in firm contact, the soil between the pile and the pile group behave as a unit.

At small pile spacings, a block type failure mechanism may prevail, whereas individual pile failure may occur at larger pile spacings. It is necessary to check for failure mechanisms and design for the case that yields the minimum capacity.

For a pile group of width *X*, length *Y*, and depth *Z*, as shown in **Figure 8-33**, the bearing capacity for block failure is given by:

$$Q_g = (2X + 2Y)Z\overline{S_u} + XYN_cS_u \qquad (8\text{-}64)$$

in which:

for (8-65)

$$\frac{Z}{X} \leq 2.5:$$

$$N_c = 5\left(1 + \frac{0.2X}{Y}\right)\left(1 + \frac{0.2Z}{X}\right)$$

for

(8-66)

$$\frac{Z}{X} > 2.5:$$

$$N_c = 7.5\left(1 + \frac{0.2X}{Y}\right)$$

S_u	=	average undrained shear strength along the depth of penetration of the piles (KSF)
S_u	=	undrained shear strength at the base of the group (KSF)

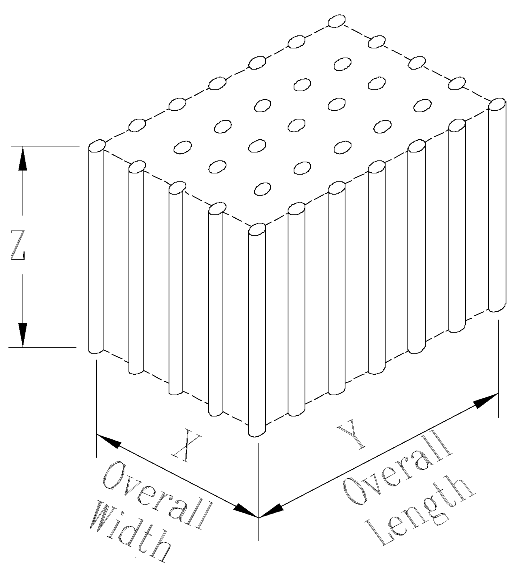


Figure 8-33 Pile Group Acting as a Block Foundation.

8.12.4.9 Uplift Resistance of Single Piles

Uplift on single piles shall be evaluated when tensile forces are present. The factored nominal tensile resistance of the pile due to soil failure shall be greater than the factored pile loads. The uplift resistance of a single pile should be estimated in a manner similar to that for estimating the skin friction resistance of piles in compression specified in **WSDOT GDM Section 8.12.4.7.5**.

Factored uplift resistance in KIPS shall be taken as:

$$R_R = \varphi_{up} R_n = \varphi_{up} R_s \tag{8-67}$$

where:

R_S = nominal uplift resistance due to side resistance (KIPS)

φ_{up} = resistance factor for uplift resistance specified in **Table 8-8**

Note that the resistance factor for uplift already is reduced to 80% of the resistance factor for static skin friction resistance. Therefore, the skin friction resistance estimated based on **Section 8.12.4.7.5** does not need to be reduced to account for uplift effects on skin friction.

Uplift resistance of single piles may be determined by static load test. If a static uplift test is to be performed, it shall follow the procedures specified in ASTM 3689. Static uplift tests should be evaluated using a modified Davisson Method as described in **Hannigan et al. (2005)**.

If pile load uplift test(s) are conducted, they should be used to calibrate the static analysis method (i.e., back calculate soil properties) to adjust the calculated uplift resistance for variations in the stratigraphy. Based on the calculated uplift resistance using the pile load test results, the minimum penetration criterion to obtain the desired uplift resistance is established.

8.12.4.10 Uplift Resistance of Pile Groups

The nominal uplift resistance of pile groups shall be evaluated when the foundation is subjected to uplift loads. Pile group factored uplift resistance, in KIPS, shall be taken as:

$$R_R = \varphi R_n = \varphi_{ug} R_{ug} \tag{8-68}$$

where:

φ_{ug} = resistance factor specified in **Table 8-8**

R_{ug} = nominal uplift resistance of the pile group (KIPS)

The uplift resistance, R_{ug}, of a pile group shall be taken as the lesser of:

- The sum of the individual pile uplift resistance, or
- The uplift resistance of the pile group considered as a block.

For pile groups in cohesionless soil, the weight of the block that will be uplifted shall be determined using a spread of load of 1H in 4V from the base of the pile group taken from **Figure 8-34**. Buoyant unit weights shall be used for soil below the groundwater level.

In cohesive soils, the block used to resist uplift in undrained shear shall be taken from **Figure 8-35**. The nominal group uplift resistance may be taken as:

$$R_n = R_{ug} = (2XZ + 2YZ)\bar{S}_u + W_g \tag{8-69}$$

where:

X	=	width of the group, as shown in **Figure 8-35** (FT)
Y	=	length of the group, as shown in **Figure 8-35** (FT)
Z	=	depth of the block of soil below pile cap taken from **Figure 8-35** (FT)
$\bar{S}_u$	=	average undrained shear strength along the sides of the pile group (KSF)
W_g	=	weight of the block of soil, piles, and pile cap (KIPS)

The resistance factor for the nominal group uplift resistance, R_{ug}, determined as the sum of the individual pile resistance, shall be taken as the same as that for the uplift resistance of single piles as specified in **Table 8-8**. The resistance factor for the uplift resistance of the pile group considered as a block shall be taken as specified in **Table 8-8** for pile groups in clay and in sand.

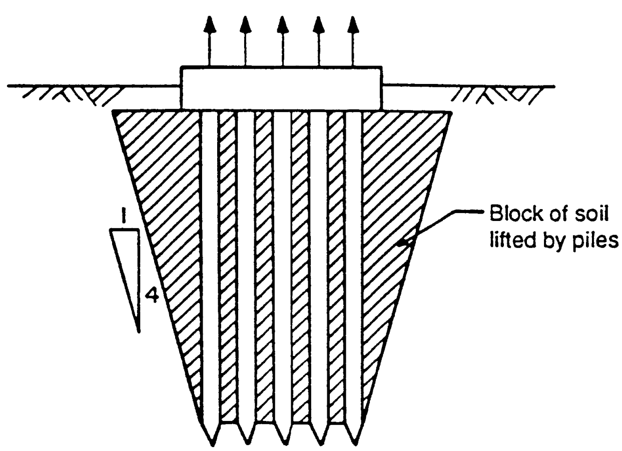


Figure 8-34 Uplift of Group of Closely Spaced Piles in Cohesionless Soils after Tomlinson (1987).

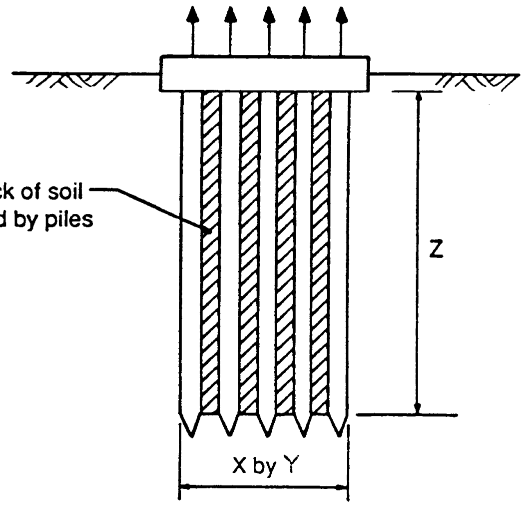


Figure 8-35 Uplift of Group of Piles in Cohesive Soils after Tomlinson (1987).

8.12.4.11 Nominal Horizontal Resistance of Pile Foundations

The nominal resistance of pile foundations to horizontal loads shall be evaluated based on both geomaterial and structural properties. The horizontal soil resistance along the piles should be modeled using P-y curves developed for the soils at the site, or as appropriate, strain wedge theory **(Norris, 1986; Ashour, et al., 1998)**, as specified in **WSDOT GDM Section 8.12.2.5**.

The applied loads shall be factored loads and they must include both horizontal and axial loads. The analysis may be performed on a representative single pile with the appropriate pile top boundary condition or on the entire pile group. If P-y curves are used, they shall be modified for group effects. The P-multipliers in **Table 8-21** should be used to modify the curves. If strain wedge theory is used, P-multipliers shall not be used, but group effects shall be addressed through evaluation of the overlap between shear zones formed due to the passive wedge that develops in front of each pile in the group as lateral deflection increases. If the pile cap will always be embedded, the P-y horizontal resistance of the soil on the cap face may be included in the horizontal resistance.

The minimum penetration of the piles below ground (see **WSDOT GDM Section 8.12.6**) required in the contract should be established such that fixity is obtained. For this determination, the loads applied to the pile are factored, and a soil resistance factor of 1.0 shall be used as specified in **Table 8-8**.

If fixity cannot be obtained, additional piles should be added, larger diameter piles used if feasible to drive them to the required depth, or a wider spacing of piles in the group should be considered to provide the necessary lateral resistance. Batter piles should be added as a last resort. Batter piles should not be used if downdrag is anticipated. The design procedure in this case should take into consideration the lack of fixity of the pile.

The strength limit state for lateral resistance is only structural (see Sections 5 and 6 of the AASHTO LRFD Bridge Design Specifications for structural limit state design requirements), though the determination of pile fixity is the result of soil-structure interaction. A failure of the soil does not occur; the soil will continue to displace at constant or slightly increasing resistance. Failure occurs when the pile reaches the structural limit state, and this limit state is reached, in the general case, when the nominal combined bending and axial resistance is reached.

If the lateral resistance of the soil in front of the pile cap is included in the horizontal resistance of the foundation, the effect of soil disturbance resulting from construction of the pile cap should be considered. In such cases, the passive resistance may need to be reduced to account for the effects of disturbance.

8.12.5 Extreme Event Limit State Design of Pile Foundations

For the applicable factored loads (see **AASHTO LRFD Bridge Design Specifications, Section 3**) for each extreme event limit state, the pile foundations shall be designed to have adequate factored axial and lateral resistance. For seismic design, all soil within and above liquefiable zones, shall not be considered to contribute axial compressive resistance. Downdrag resulting from liquefaction induced settlement shall be determined as specified in **WSDOT GDM Section 8.6.2** and included in the loads applied to the foundation. Static downdrag loads should not be combined with seismic downdrag loads due to liquefaction.

In general, the available factored geotechnical resistance should be greater than the factored loads applied to the pile, including the downdrag, at the extreme event limit state. The pile foundation shall be designed to structurally resist the downdrag plus structure loads.

Pile design for liquefaction downdrag is illustrated in **Figure 8-36**, where,

R_{Sdd} =	skin friction which must be overcome during driving through downdrag zone
$Q_p = (\Sigma\gamma_i Q_i)$ =	factored load per pile, excluding downdrag load
DD =	downdrag load per pile
$D_{est.}$ =	estimated pile length needed to obtain desired nominal resistance per pile
φ_{seis} =	resistance factor for seismic conditions
γ_p =	load factor for downdrag

The nominal bearing resistance of the pile needed to resist the factored loads, including downdrag, is therefore,

$$R_n = (\Sigma\gamma_i Q_i)/\varphi_{seis} + \gamma_p DD/\varphi_{seis} \quad (8\text{-}70)$$

The total driving resistance, R_{ndr}, needed to obtain R_n, accounting for the skin friction that must be overcome during pile driving that does not contribute to the design resistance of the pile, is as follows:

$$R_{ndr} = R_{Sdd} + R_n \quad (8\text{-}71)$$

Note that R_{Sdd} remains unfactored in this analysis to determine R_{ndr}.

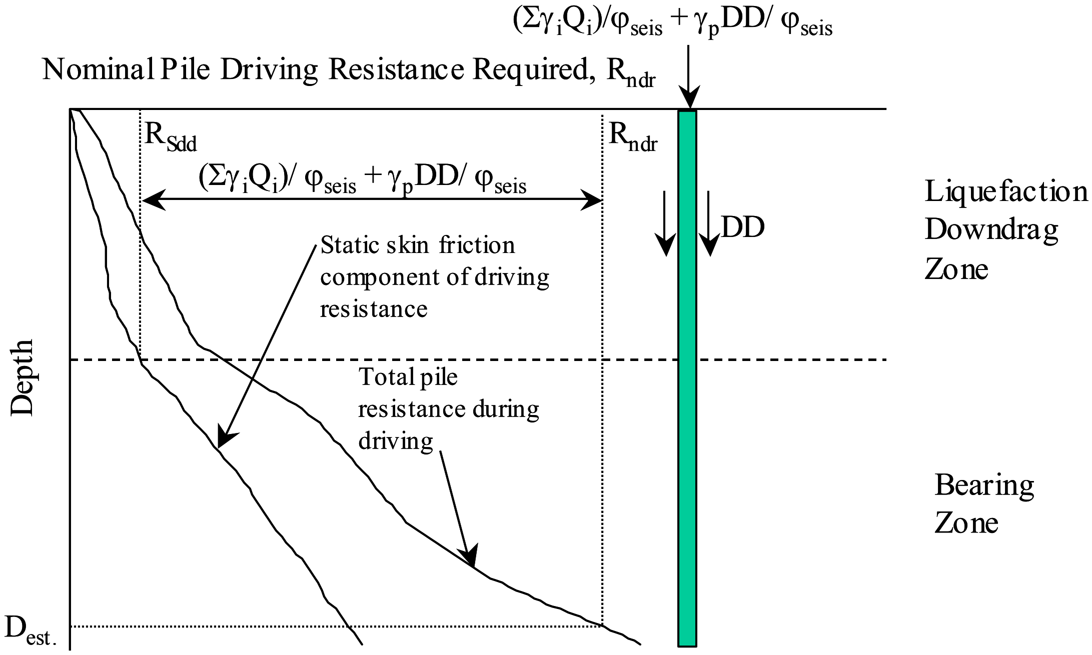


Figure 8-36 Design of pile foundations for liquefaction downdrag.

In the instance where it is not possible to obtain adequate geotechnical resistance below the lowest layer contributing to downdrag (e.g., friction piles) to fully resist the downdrag, or if it is anticipated that significant deformation will be required to mobilize the geotechnical resistance needed to resist the factored loads including the downdrag load, the structure should be designed to tolerate the settlement resulting from the downdrag and the other applied loads in accordance with **WSDOT GDM Section 8.12.3.4**.

The static analysis procedures in **WSDOT GDM Section 8.12.4.7.5** may be used to estimate the available pile resistance to withstand the downdrag plus structure loads to estimate pile lengths required to achieve the required bearing resistance. For this calculation, it should be assumed that the soil subject to downdrag still contributes overburden stress to the soil below the downdrag zone.

Resistance may also be estimated using a dynamic method per **WSDOT GDM Section 8.12.4.7.2**, provided the skin friction resistance within the zone contributing to downdrag is subtracted from the resistance determined from the dynamic method during pile installation. The skin friction resistance within the zone contributing to downdrag may be estimated using the static analysis methods specified in **WSDOT GDM Section 8.12.4.7.5**, from signal matching analysis, or from pile load test results. Note that the static analysis method may have a bias, on average over or under predicting the skin friction. The bias of the method selected to estimate the skin friction within and above the downdrag zone should be taken into account as described in **WSDOT GDM Section 8.12.4.2**.

The pile foundation shall also be designed to resist the horizontal force resulting from lateral spreading, if applicable, or the liquefiable soil shall be improved to prevent liquefaction and lateral spreading. For lateral soil resistance of the pile foundation, the P-y curve soil parameters should be reduced to account for liquefaction. To determine the amount of reduction, the duration of strong shaking and the ability of the soil to fully develop a liquefied condition during the period of strong shaking should be considered.

The force resulting from lateral spreading should be calculated as described in **WSDOT GDM Chapter 6**. In general, the lateral spreading force should not be combined with the seismic forces, except possibly for very large magnitude, long duration earthquakes (e.g., magnitude 8.0+ earthquakes). See **WSDOT GDM Chapter 6** for additional guidance regarding this issue.

Regarding the reduction of soil strength and stiffness parameters to account for liquefaction, fully liquefied soil may be treated as a soft clay, using residual strength parameters from **Seed and Harder (1990)**, assuming the strain required to mobilize 50% of the ultimate resistance to be equal to 0.02. Alternatively, the soil can be treated as a very loose sand, or computer programs that contain theoretical algorithms to generate the pore pressures induced by liquefaction and can thereby calculate directly the liquefied soil stiffness parameters may be used. Regardless of the method selected good engineering judgment will be necessary. The geotechnical designer should be aware that use of the soft clay model to simulate the as-liquefied soil may result in a stiffer response than the sand model for the un-liquefied condition at low loads or displacements due to the difference in the shape of the P-y curves.

Since the timing of the full seismic forces and the development of fully liquefied conditions is uncertain, and also depends on the length of time over which strong shaking occurs, typical practice is to bracket the stiffness by determining the load distribution as specified in **WSDOT GDM Section 8.6.1**, including the full seismic forces, using both un-liquefied and liquefied parameters (both the soil parameters and the loads are unfactored at this point). Once the loads have been distributed, then factor the loads and resistances as necessary to evaluate each potential failure mechanism for all aspects of the structural and geotechnical design. For the evaluation of fixity of the pile for extreme event limit state design resistance factor is 1.0 and should be applied to the soil stiffness values.

When designing for scour at the extreme event limit state, the pile foundation design shall be conducted as described in **WSDOT GDM Section 8.12.4.5**, except that the check flood per the AASHTO Bridge Design Specifications and resistance factors consistent with **WSDOT GDM Section 8.10.1** shall be used.

8.12.6 Determination of Minimum Pile Penetration

The minimum pile penetration, if required for the particular site conditions and loading, shall be based on the maximum depth needed to meet the following requirements as applicable:

- Single and pile group settlement (service limit state)
- Lateral deflection (service limit state)
- Uplift (strength limit state)
- Depth into bearing soils needed to resist downdrag loads resulting from static consolidation stresses on soft soil or downdrag loads due to liquefaction (strength and extreme event limit state, respectively)

- Depth into bearing soils needed to provide adequate pile axial (compression and uplift) and lateral resistance after scour (strength and extreme event limit states)
- Nominal soil shear resistance and fixity for resisting the applied lateral loads to the foundation (strength limit state)
- Axial uplift, and lateral resistance to resist extreme event limit state loads

The contract documents should indicate the minimum pile penetration (if applicable) as determined above. The contract documents should also include the required nominal axial compressive resistance, R_{ndr} (see **WSDOT GDM Section 8.12.7**) and the method by which this resistance will be verified (if applicable) such that the resistance factor(s) used for design are consistent with the construction field verification methods of nominal axial compressive pile resistance.

A minimum pile penetration should only be specified if necessary to insure that all of the applicable limit states are met. A minimum pile penetration should not be specified to meet axial compression resistance (i.e., bearing), unless field verification of the pile nominal bearing resistance is not performed (see **WSDOT GDM Section 8.12.4.7**).

8.12.7 Determination of R_{ndr} Used to Establish Contract Driving Criteria for Bearing

The value of R_{ndr} used for the construction of the pile foundation to establish the driving criteria to obtain the design bearing resistance shall be the value that meets or exceeds the following limit states, as applicable:

- Strength limit state compression resistance (**WSDOT GDM Sections 8.12.4.7.2, 8.12.4.7.3, or 8.12.4.7.4** if bearing resistance is determined in the field using a dynamic method, or **Section 8.12.4.7.5** if bearing resistance is not verified in the field and the pile is driven to a specified tip elevation for bearing)
- Strength limit state compression resistance, including downdrag (**WSDOT GDM Section 8.12.4.6**)
- Strength limit state compression resistance, accounting for scour (**WSDOT GDM Section 8.12.4.5**)
- Extreme event limit state compression resistance for seismic (**WSDOT GDM Section 8.12.5**)
- Extreme event limit state compression resistance for scour (**WSDOT GDM Section 8.12.5**)

8.12.8 Pile Drivability Analysis

The establishment of the installation criteria for driven piles should include a drivability analysis. The drivability analysis shall be performed by the Engineer using a wave equation analysis, and the driving stresses (σ_{dr}) anywhere in the pile determined from the analysis shall be less than the following limits:

Steel Piles, compression and tension:

- $\sigma_{dr} = \varphi_{da} 0.9 f_y$ (8-72)

where, f_y is the yield strength of the steel. and φ_{da} is the resistance factor as specified in **Table 8-8**.

Concrete piles:

- In compression, $\sigma_{dr} = \varphi_{da} 0.85 f'_c$ (8-73)

- In tension, considering only the steel reinforcement,
  $\sigma_{dr} = \varphi_{da} 0.7 f_y$ (8-74)

where, is the unconfined compressive strength of the concrete, and f_y is the yield strength of the steel reinforcement.

Prestressed concrete piles, normal environments:

- In compression, $\sigma_{dr} = \varphi_{da} \left(0.85 f'_c - f_{pe}\right)$ (8-75)

- In tension, $\sigma_{dr} = \varphi_{da} \left(3\sqrt{f'_c} + f_{pe}\right)$ (8-76)

where, f'_c and f_{pe} must be in PSI, and the resulting maximum stress is also in PSI.

Prestressed concrete piles, severe corrosive environments:

- In tension, $\sigma_{dr} = \varphi_{da} f_{pe}$ (8-77)

Timber piles, in compression and tension:

- $\sigma_{dr} = \varphi_{da}(3F_{co})$ (8-78)

where, F_{co} is the base resistance of wood in compression parallel to the grain.

For routine pile installation applications where significant local experience can be applied to keep the risk of pile installation problems low, a project specific drivability analysis using the wave equation may be waived.

This drivability analysis shall be based on the maximum driving resistance needed:

- To obtain minimum penetration requirements per **WSDOT GDM Section 8.12.6**,
- To overcome resistance of soil that cannot be counted upon to provide axial or lateral resistance throughout the design life of the structure (e.g., material subject to scour, or material subject to downdrag), and
- To obtain the required nominal bearing resistance.

Wave equation analyses should be conducted during design using a range of likely hammer/pile combinations, considering the soil and installation conditions at the foundation site. See **WSDOT GDM Section 8.12.4.7.3** for additional considerations for conducting wave equation analyses. These analyses should be used to assess feasibility of the proposed foundation system and to establish installation criteria with regard to driving stresses to limit driving stresses to acceptable levels. For routine pile installation applications (e.g., smaller diameter, low nominal resistance piles), the development of installation criteria with regard to the limitation of driving stresses (e.g., minimum or maximum ram weight, hammer size, maximum acceptable driving resistance, etc.) may be based on local experience, rather than conducting a detailed wave equation analysis that is project specific. Local experience could include previous drivability analysis results and actual pile driving experience that are applicable to the project specific situation at hand. Otherwise, a project specific drivability study should be conducted.

Drivability analyses may also be conducted as part of the project construction phase. When conducted during the construction phase, the drivability analysis shall be conducted using the contractor's proposed driving system. This information should be supplied by the contractor. This drivability analysis should be used to determine if the contractor's proposed driving system is capable of driving the pile to the maximum resistance anticipated without exceeding the factored structural resistance available (i.e., σ_{dr}).

In addition to this drivability analysis, the best approach to controlling driving stresses during pile installation is to conduct dynamic testing with signal matching to verify the accuracy of the wave equation analysis results, and to calibrate the wave equation analyses. Note that if a drivability analysis is conducted using the wave equation for acceptance of the contractor's proposed driving system, but a different method is used to develop driving resistance (i.e., blow count) criterion to obtain the specified nominal pile resistance (e.g., a driving formula), the difference in the methods regarding the predicted driving resistance should be taken into account when evaluating the contractor's driving system. For example, the wave equation analysis could indicate that the contractor's hammer can achieve the desired bearing resistance, but the driving formula could indicate the driving resistance at the required nominal bearing is too high. Such differences should be considered when setting up the driving system acceptance requirements in the contract documents.

The selection of a blow count limit is difficult because it can depend on the site soil profile, the pile type, and possibly hammer manufacturer limitations to prevent hammer damage. In general, blow counts greater than 10 to 15 blows per inch should be used with care, particularly with concrete or timber piles. In cases where the driving is easy until near the end of driving, a higher blow count may sometimes be satisfactory, but if a high blow count is required over a large percentage of the depth, even 10 blows per inch may be too large.

8.12.9 Test Piles

Test piles should be driven at several locations on the site to establish order length. These test piles should be driven after the driving criteria have been established if dynamic measurements are not taken.

If dynamic measurements during driving are taken, both order lengths and driving criteria should be established after the test pile(s) are driven. Dynamic measurements obtained during test pile driving, signal matching analyses, and wave equation analyses should be used to determine the driving criteria (bearing requirements) as described in **WSDOT GDM Sections 8.12.4.7.1**, **8.12.4.7.2**, and **8.12.4.7.3**.

8.13 Drilled Shaft Foundation Design

Figure 8-37 provides a flowchart that illustrates the design process, and interaction required between structural and geotechnical engineers, needed to complete a drilled shaft foundation design. ST denotes steps usually completed by the Structural Designer, while GT denotes those steps normally completed by the Geotechnical Designer.

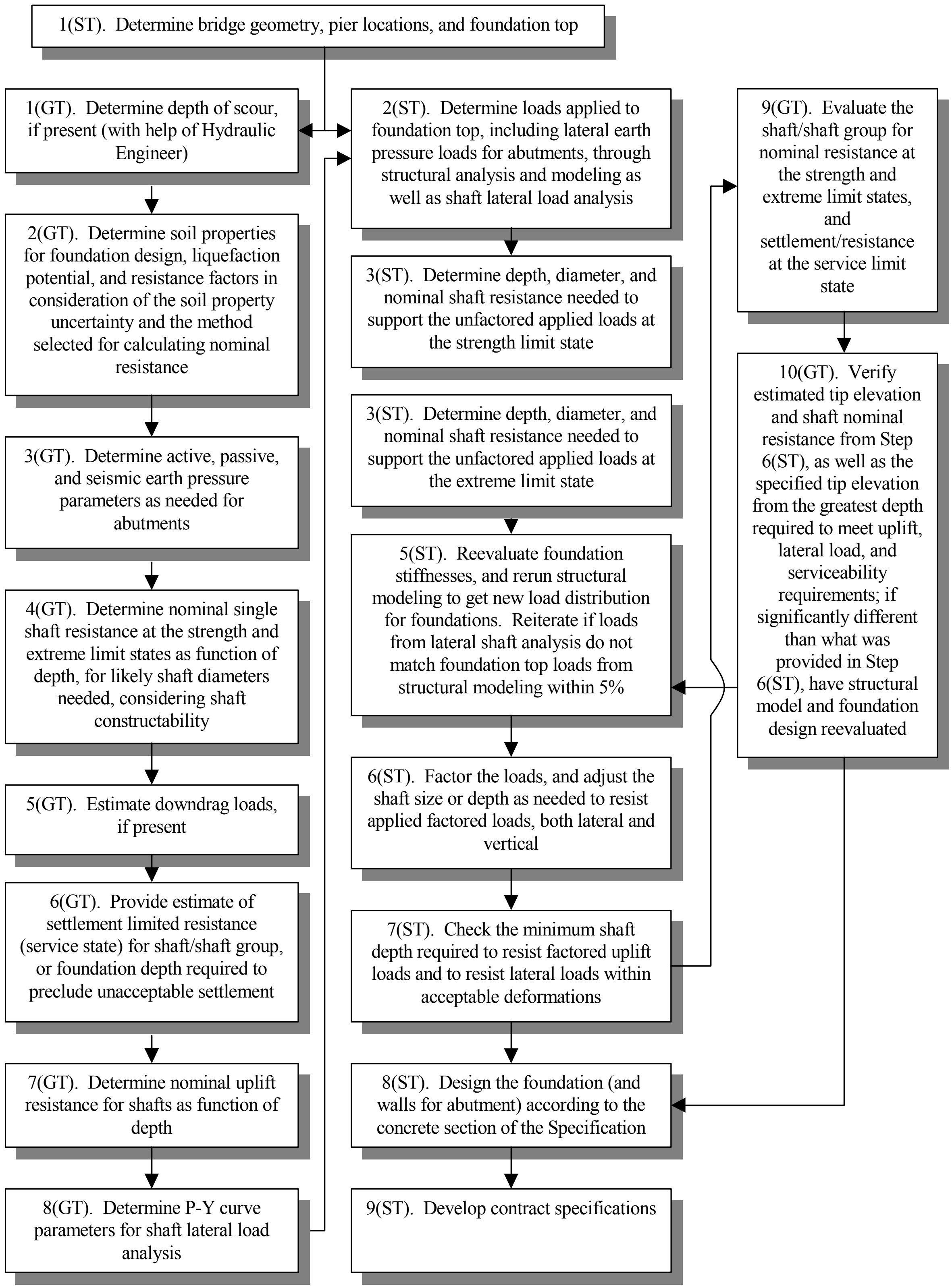


Figure 8-37 Design flowchart for drill shaft foundation design.

8.13.1 Loads and Load Factor Application to Drilled Shaft Design

Figures 8-25 and 8-26 provide definitions and typical locations of the forces and moments that act on deep foundations such as drilled shafts. **Table 8-19** identifies when to use maximum or minimum load factors for the various modes of failure for the shaft (bearing capacity, uplift, and lateral loading) for each force, for the strength limit state.

The loads and load factors to be used in pile foundation design shall be as specified in Section 3 of the AASHTO LRFD Bridge Design Specifications. Computational assumptions that shall be used in determining individual shaft loads are described in Section 4 of the AASHTO LRFD specifications.

8.13.2 General Considerations for Drilled Shaft Geotechnical Design

The provisions of **WSDOT GDM Section 8.13** and all subsections shall apply to the design of drilled shafts. Throughout these provisions, the use of the term “drilled shaft” shall be interpreted to mean a shaft constructed using either drilling or casing plus excavation equipment and related technology. These provisions shall also apply to shafts that are constructed using casing advancers that twist or rotate casings into the ground concurrent with excavation rather than drilling. The provisions of this section are not applicable to drilled piles installed with continuous flight augers that are concreted as the auger is being extracted (e.g., this section does not apply to the design of augercast piles).

Drilled shafts are classified according to their primary mechanism for deriving resistance either as friction shafts, i.e., shafts transferring load primarily by side resistance, or end-bearing shafts, i.e., shafts transferring load primarily by tip resistance.

Shaft designs should be reviewed for constructability prior to advertising the project for bids.

8.13.2.1 Drilled Shaft Resistance

Drilled shafts shall be designed to have adequate axial and structural resistance, tolerable settlements, and tolerable lateral displacements. The axial resistance of drilled shafts shall be determined through a suitable combination of subsurface investigations, laboratory and/or in-situ tests, analytical methods, and load tests, with reference to the history of past performance. The following additional issues shall be addressed as applicable:

- The difference between the resistance of a single shaft and that of a group of shafts;
- The resistance of the underlying strata to support the load of the shaft group;
- The effects of constructing the shaft(s) on adjacent structures;
- The possibility of scour and its effect; and
- The transmission of forces, such as downdrag forces, from consolidating soil.
- Minimum shaft penetration necessary to satisfy the requirements caused by uplift, scour, downdrag, settlement, liquefaction, lateral loads and seismic conditions.
- Satisfactory behavior under service loads.
- Long-term durability of the shaft in service (i.e. corrosion and deterioration).

8.13.2.2 Effect of Drilled Shaft Installation Technique on Resistance

The method of construction affects the shaft axial and lateral resistance. The shaft design parameters shall take into account the likely construction methodologies used to install the shaft. The performance of drilled shaft foundations can be greatly affected by the method of construction, particularly side resistance. The designer should consider the effects of ground and groundwater conditions on shaft construction operations and delineate, where necessary, the general method of construction to be followed to ensure the expected performance. Because shafts derive their resistance from side and tip resistance, which is a function of the condition of the materials in direct contact with the shaft, it is important that the construction procedures be consistent with the material conditions assumed in the design. Softening, loosening, or other changes in soil and rock conditions caused by the construction method could result in a reduction in shaft resistance and an increase in shaft displacement. Therefore, evaluation of the effects of the shaft construction procedure on resistance should be considered an inherent aspect of the design. Use of slurries, varying shaft diameters, and post grouting can also affect shaft resistance.

Soil parameters should be varied systematically to model the range of anticipated conditions. Both vertical and lateral resistance should be evaluated in this manner.

Procedures that may affect axial or lateral shaft resistance include, but are not limited to, the following:

- Artificial socket roughening, if included in the design nominal axial resistance assumptions.
- Removal of temporary casing where the design is dependent on concrete-to-soil adhesion.
- The use of permanent casing.
- Use of tooling that produces a uniform cross-section where the design of the shaft to resist lateral loads cannot tolerate the change in stiffness if telescoped casing is used.

It should be recognized that the design procedures provided in these specifications assume compliance to construction specifications that will produce a high quality shaft. Performance criteria should be included in the construction specifications that require:

- Shaft bottom cleanout criteria,
- Appropriate means to prevent side wall movement or failure (caving) such as temporary casing, slurry, or a combination of the two,
- Slurry maintenance requirements including minimum slurry head requirements, slurry testing requirements, and maximum time the shaft may be left open before concrete placement.

If for some reason one or more of these performance criteria are not met, the design should be reevaluated and the shaft repaired or replaced as necessary.

8.13.2.3 Shaft Spacing

If the center-to-center spacing of drilled shafts is less than 4.0 diameters, the interaction effects between adjacent shafts shall be evaluated. If the center-to-center spacing of drilled shafts is less than 6.0 diameters, the sequence of construction should be specified in the contract documents. Larger spacing may be required to preserve shaft excavation stability or to prevent communication between shafts during excavation and concrete placement. Shaft spacing may be decreased if casing construction methods are required to maintain excavation stability and to prevent interaction between adjacent shafts.

8.13.2.4 Shaft Diameter and Enlarged Bases

If the shaft is to be manually inspected, the shaft diameter should not be less than 30.0 IN. The diameter of columns supported by shafts should be smaller than or equal to the diameter of the drilled shaft.

If the shaft and the column are the same diameter, it should be recognized that the placement tolerance of drilled shafts is such that it will likely affect the column location. The shaft and column diameter should be determined based on the shaft placement tolerance, column and shaft reinforcing clearances, and the constructability of placing the column reinforcing in the shaft. A horizontal construction joint in the shaft at the bottom of the column reinforcing will facilitate constructability. Making allowance for the tolerance where the column connects with the superstructure, which could affect column alignment, can also accommodate this shaft construction tolerance.

Nominal shaft diameters used for both geotechnical and structural design of shafts should be selected based on available diameter sizes.

In drilling rock sockets, it is common to use casing through the soil zone to temporarily support the soil to prevent cave-in, allow inspection and to produce a seal along the soil-rock contact to minimize infiltration of groundwater into the socket. Depending on the method of excavation, the diameter of the rock socket may need to be sized at least 6 inches smaller than the nominal casing size to permit seating of casing and insertion of rock drilling equipment.

In stiff cohesive soils, an enlarged base (bell, or underream) may be used at the shaft tip to increase the tip bearing area to reduce the unit end bearing pressure or to provide additional resistance to uplift loads.

Where the bottom of the drilled hole is dry, cleaned and inspected prior to concrete placement, the entire base area may be taken as effective in transferring load.

Where practical, consideration should be given to extension of the shaft to a greater depth to avoid the difficulty and expense of excavation for enlarged bases.

8.13.2.5 Battered Shafts

Battered shafts should be avoided. Where increased lateral resistance is needed, consideration should be given to increasing the shaft diameter or increasing the number of shafts. Due to problems associated with hole stability during excavation, installation, and with removal of casing and with installation of the rebar cage and concrete placement, construction of battered shafts is very difficult.

8.13.2.6 Nearby Structures

Where shaft foundations are placed adjacent to existing structures, the influence of the existing structure on the behavior of the foundation, and the effect of the foundation on the existing structures, including vibration effects due to casing installation, should be investigated. In addition, the impact of caving soils during shaft excavation on the stability of foundations supporting adjacent structures should be evaluated. For existing structure foundations that are adjacent to the proposed shaft foundation, and if a shaft excavation cave-in could compromise the existing foundation in terms of stability or increased deformation, the design should require that casing be advanced as the shaft excavation proceeds.

8.13.3 Service Limit State Design of Drilled Shafts

Drilled shaft foundations shall be designed at the service limit state to meet the tolerable movements for the structure being supported in accordance with **WSDOT GDM Section 8.6.5.1**.

Service limit state design of drilled shaft foundations includes the evaluation of settlement due to static loads, and downdrag loads if present, overall stability, lateral squeeze, and lateral deformation. Overall stability of a shaft supported foundation shall be evaluated where:

- The foundation is placed through an embankment,
- The pile foundation is located on, near or within a slope,
- The possibility of loss of foundation support through erosion or scour exists, or
- Bearing strata are significantly inclined.

In general, it is not desirable to subject the shaft foundation to unbalance lateral loading caused by lack of overall stability or caused by lateral squeeze. The unbalanced lateral forces should be mitigated through stabilization measures, if possible.

Lateral analysis of shaft foundations is conducted to establish the load distribution between the superstructure and foundations for all limit states, and to estimate the deformation in the foundation that will occur due to those loads. This section only addresses the evaluation of the lateral deformation of the foundation resulting from the distributed loads.

8.13.3.1 Settlement

Settlement of single shafts and shaft groups shall be investigated, including both short-term and long-term settlement.

8.13.3.1.1 Settlement of Single Shafts

The settlement of single-drilled shafts shall be estimated in consideration of:

- Short-term settlement,
- Consolidation settlement if constructed in or above cohesive soils, and
- Axial compression of the shaft.

The normalized load-settlement curves shown in **Figures 8-38 through 8-41** should be used to limit the nominal shaft axial resistance computed as specified for the strength limit state in **WSDOT GDM Section 8.13.4.4** for service limit state tolerable movements. Consistent values of normalized settlement shall be used for limiting the base and side resistance when using these figures. These curves do not include consideration of long-term consolidation settlement for shafts in cohesive soils. Long-term settlement should be computed according to **WSDOT GDM Section 8.12.3.1** using the equivalent footing method and added to the short-term settlements estimated using **Figures 8-38 though 8-41**.

Other methods for evaluating shaft settlements that may be used are found in **O'Neill and Reese (1999)**.

O'Neill and Reese (1999) have summarized load-settlement data for drilled shafts in dimensionless form, as shown in **Figures 8-38 through 8-41**. **Figures 8-38 and 8-39** show the load-settlement curves in side resistance and in end bearing for shafts in cohesive soils. **Figures 8-40 and 8-41** are similar curves for shafts in cohesionless soils. These curves should be used for estimating short-term settlements of drilled shafts.

The designer should exercise judgment relative to whether the trend line, one of the limits, or some relation in between should be used from **Figures 8-38 through 8-41**.

The values of the load-settlement curves in side resistance were obtained at different depths, taking into account elastic shortening of the shaft. Although elastic shortening may be small in relatively short shafts, it may be quite substantial in longer shafts. The amount of elastic shortening in drilled shafts varies with depth. **O'Neill and Reese (1999)** have described an approximate procedure for estimating the elastic shortening of long drilled shafts.

Settlements induced by loads in end bearing are different for shafts in cohesionless soils and in cohesive soils. Although drilled shafts in cohesive soils typically have a well-defined break in a load-displacement curve, shafts in cohesionless soils often have no well-defined failure at any displacement. The resistance of drilled shafts in cohesionless soils continues to increase as the settlement increases beyond 5 percent of the base diameter. The shaft end bearing R_p is typically fully mobilized at displacements of 2 to 5 percent of the base diameter for shafts in cohesive soils. The unit end bearing resistance for the strength limit state (see **WSDOT GDM Sections 8.13.4.4.1(b) and 8.13.4.4.2(b)**) is defined as the bearing pressure required to cause settlement equal to 5 percent of the shaft diameter, even though this does not correspond to complete failure of the soil beneath the base of the shaft.

The curves in **Figures 8-38 and 8-40** also show the settlements at which the side resistance is mobilized. The shaft skin friction R_S is typically fully mobilized at displacements of 0.2 percent to 0.8 percent of the shaft diameter for shafts in cohesive soils. For shafts in cohesionless soils, this value is 0.1 percent to 1.0 percent.

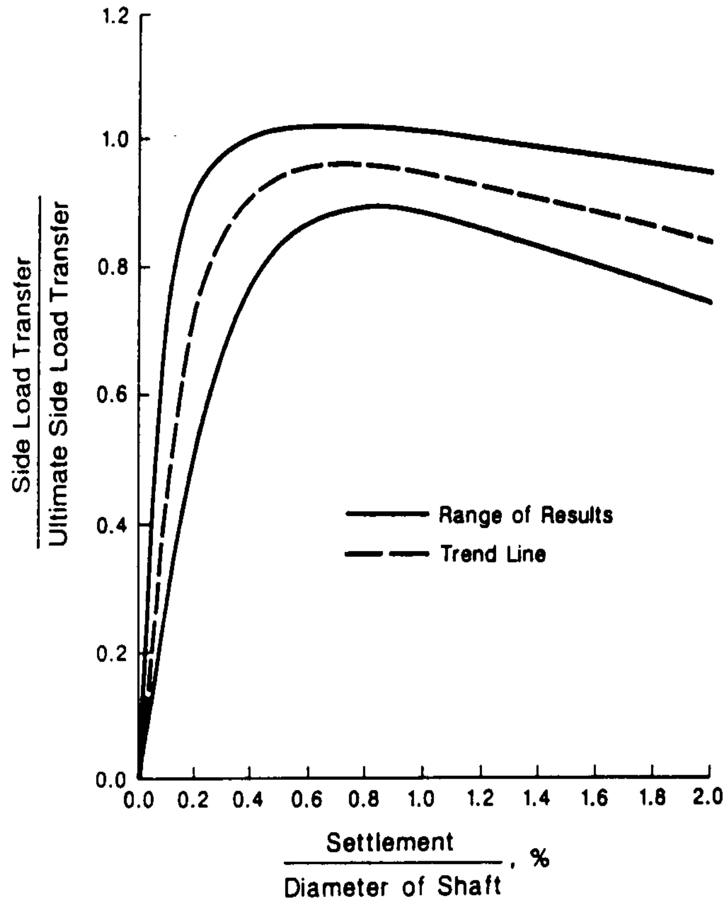


Figure 8-38 Normalized Load Transfer in Side Resistance Versus Settlement in Cohesive Soils (from O'Neill & Reese, 1999).

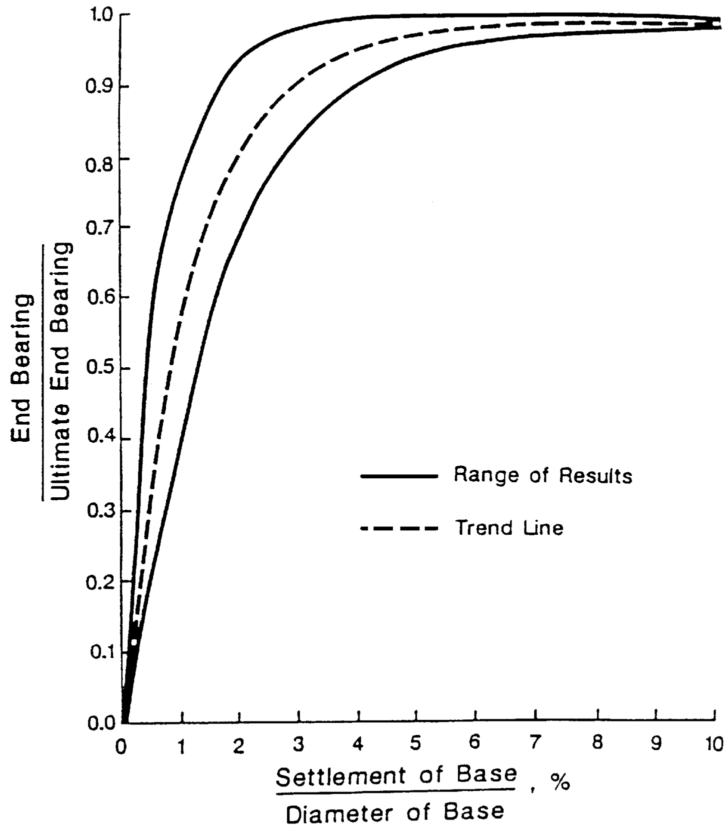


Figure 8-39 Normalized Load Transfer in End Bearing Versus Settlement in Cohesive Soils (from O'Neill & Reese, 1999).

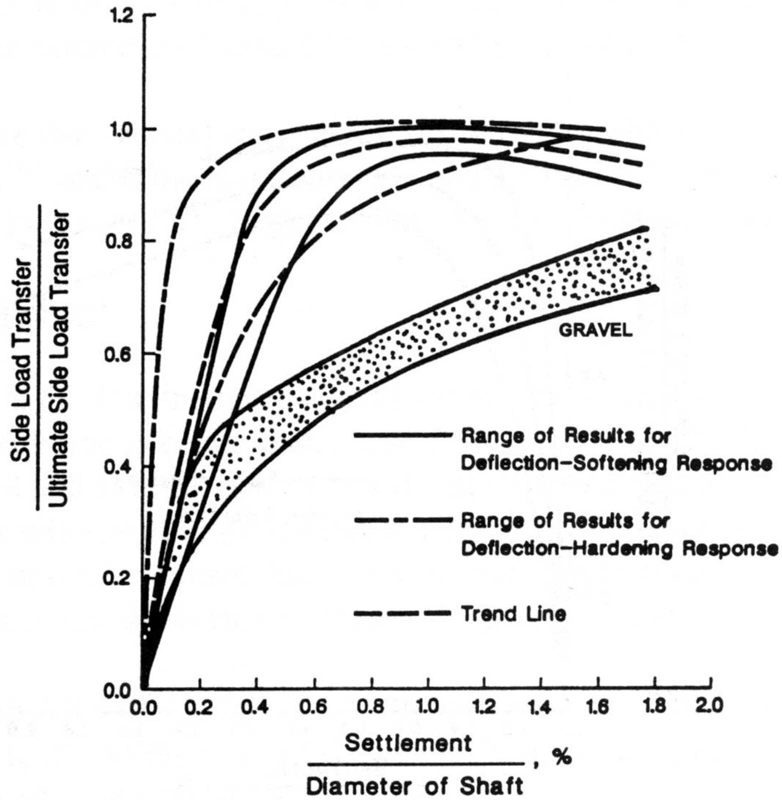


Figure 8-40 Normalized Load Transfer in Side Resistance Versus Settlement in Cohesionless Soils (from O'Neill & Reese, 1999).

The deflection-softening response in **Figure 8-40** typically applies to cemented or partially cemented soils, or other soils that exhibit brittle behavior and have low residual shear strengths at larger deformations. Note that the trend line for sands is a reasonable approximation for either the deflection-softening or deflection-hardening response.

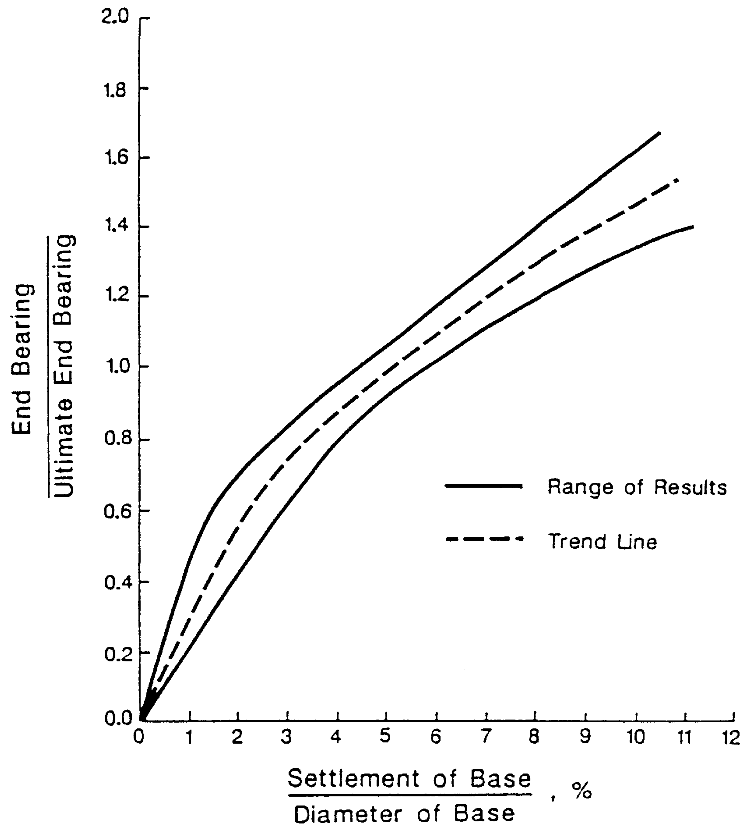


Figure 8-41 Normalized Load Transfer in End Bearing Versus Settlement in Cohesionless Soils (from O'Neill & Reese, 1999).

8.13.3.1.2 Settlement of Shafts in Intermediate Geomaterials (IGM's)

For detailed settlement estimation of shafts in IGM's, the procedures provided by **O'Neill and Reese (1999)** should be used.

IGM's are defined by **O'Neill and Reese (1999)** as follows:

- Cohesive IGM – clay shales or mudstones with an S_u of 5 to 50 KSF, and
- Cohesionless – granular tills or granular residual soils with $N1_{60}$ greater than 50 blows/ft.

8.13.3.1.3 Settlement of Shaft Groups

The provisions of **WSDOT GDM Section 8.12.3.1** shall apply. Shaft group effect shall be considered for groups of 2 or more.

O'Neill and Reese (1999) summarize various studies on the effects of shaft group behavior. These studies were for groups that consisted of 1 x 2 to 3 x 3 shafts. These studies suggest that group effects are relatively unimportant for shaft center-to-center spacing of 5D or greater.

8.13.3.2 Horizontal Movement of Shafts and Shaft Groups

The provisions of **WSDOT GDM Section 8.12.3.3** shall apply.

8.13.3.3 Overall Stability

The provisions of **WSDOT GDM Section 8.6.5.2** shall apply.

8.12.3.4 Settlement Due to Downdrag

The provisions of **WSDOT GDM Section 8.12.3.4** shall apply, except that available shaft resistance to withstand the downdrag plus structure loads shall be estimated as described in **WSDOT GDM Section 8.13.4.4**, and the skin friction within and above the downdrag zone shall be ignored.

8.13.3.5 Lateral Squeeze

The provisions of **WSDOT GDM Section 8.12.3.5** shall apply.

8.13.4 Strength Limit State Geotechnical Design of Drilled Shafts

The nominal shaft geotechnical resistances that shall be evaluated at the strength limit state include:

- Axial compression resistance,
- Axial uplift resistance,
- Punching of shafts through strong soil into a weaker layer,
- Lateral geotechnical resistance of soil and rock strata,
- Resistance when scour occurs, and
- Axial resistance when downdrag occurs.

8.13.4.1 Groundwater Table and Buoyancy

The applicable provisions of **WSDOT GDM Section 8.12.4.4** shall be used.

8.13.4.2 Scour

The effect of scour shall be considered in the determination of the shaft penetration. Resistance after scour shall be based on the applicable provisions of **WSDOT GDM Section 8.12.4.5**. The shaft foundation shall be designed so that the shaft penetration after the design scour event satisfies the required nominal axial and lateral resistance. For this calculation, it shall be assumed that the soil lost due to scour does not contribute to the overburden stress in the soil below the scour zone. The shaft foundation shall be designed to resist debris loads occurring during the flood event in addition to the loads applied from the structure.

The resistance factors are those used in the design without scour. The axial resistance of the material lost due to scour shall not be included in the shaft resistance.

8.13.4.3 Downdrag

The nominal shaft resistance available to support structure loads plus downdrag shall be estimated by considering only the positive skin and tip resistance below the lowest layer contributing to the downdrag. For this calculation, it shall be assumed that the soil contributing to downdrag does contribute to the overburden stress in the soil below the downdrag zone. In general, the available factored geotechnical resistance should be greater than the factored loads applied to the shaft, including the downdrag, at the strength limit state.

In the instance where it is not possible to obtain adequate geotechnical resistance below the lowest layer contributing to downdrag (e.g., friction shafts) to fully resist the downdrag, the structure should be designed to tolerate the settlement resulting from the downdrag and the other applied loads.

8.13.4.4 Nominal Axial Bearing Resistance of Single Drilled Shafts

The factored bearing resistance of drilled shafts, R_R, shall be determined using **equations 8-59 through 8-62**, where R_n, q_s, R_s, q_p, R_p, A_s A_p, and φ_{stat} are applied to drilled shafts rather than driven piles. The resistance factors for the skin friction and tip resistance, estimated for use with the methods identified, shall be as specified in **Table 8-11**. Note that if the shaft tip is in a different material than the shaft sides, different resistance factors that are consistent with the design method used to estimate side friction and tip resistance may be needed.

The nominal axial compression resistance of a shaft is derived from the tip resistance and/or shaft side resistance, i.e., skin friction. Both the tip and shaft resistances develop in response to foundation displacement. The maximum values of each are unlikely to occur at the same displacement, as described in **WSDOT GDM Section 8.13.3.1.1**.

For consistency in the interpretation of both static load tests (**WSDOT GDM Section 8.13.4.4.6**) and the normalized curves of **WSDOT GDM Section 8.13.3.1.1**, it is customary to establish the failure criterion at the strength limit state at a gross deflection equal to five percent of the base diameter for drilled shafts.

The methods for estimating drilled shaft resistance provided in this section (and associated subsections) should be used. Shaft strength limit state resistance methods not specifically addressed in this section for which adequate successful regional or national experience is available may be used, provided adequate information and experience is also available to develop appropriate resistance factors. **O'Neill and Reese (1999)** identify several methods for estimating the resistance of drilled shafts in cohesive and granular soils, intermediate geomaterials, and rock. Resistance factors have been developed for the methodology specified herein using a combination of calibration by fitting to previous allowable stress design (ASD) practice and reliability theory (see **Allen, 2005**, for additional details on the development of resistance factors for drilled shafts). Alternative design methods may be used in lieu of the methodology specified herein, provided that:

- The method selected consistently has been used with success on a regional or national basis,
- Significant experience is available to demonstrate that success, and,
- As a minimum, calibration by fitting to allowable stress design is conducted to determine the appropriate resistance factor, if inadequate measured data are available to assess the alternative method using reliability theory. A similar approach as described by **Allen (2005)** should be used to select the resistance factor for the alternative method.

8.13.4.4.1 Estimation of Drilled Shaft Resistance in Cohesive Soils

Drilled shafts in cohesive soils should be designed by total and effective stress methods for undrained and drained loading conditions, respectively.

8.13.4.4.1(a) Side Resistance in Cohesive Soils

The nominal unit side resistance, q_s, in KSF, for shafts in cohesive soil loaded under undrained loading conditions by the α-method shall be taken as:

$$q_s = \alpha S_u \qquad (8\text{-}79)$$

in which:

$$\alpha = 0.55 \qquad \text{for } S_u/p_a \leq 1.5 \qquad (8\text{-}80)$$

$$\alpha = 0.55 - 0.1(S_u/p_a - 1.5) \qquad 1.5 \leq S_u/p_a \leq 2.5 \qquad (8\text{-}81)$$

where:

S_u	=	undrained shear strength (KTSF)
α	=	adhesion factor (DIM)
p_a	=	atmospheric pressure (= 2.12 KSF)

The α-method is based on total stress. For effective stress methods for shafts in clay, see **O'Neill and Reese (1999)**. The adhesion factor is an empirical factor used to correlate the results of full-scale load tests with the material property or characteristic of the cohesive soil. The adhesion factor is usually related to S_u and is derived from the results of full-scale pile and drilled shaft load tests. Use of this approach presumes that the measured value of S_u is correct and that all shaft behavior resulting from construction and loading can be lumped into a single parameter. Neither presumption is strictly correct, but the approach is used due to its simplicity.

The values of α obtained from **equations 8-80 and 8-81** are considered applicable for both compression and uplift loading.

Values of α for contributing portions of shafts excavated dry in open or cased holes should be as specified in **equations 8-80 and 8-81**.

When permanent casing is used, the side resistance should be adjusted with consideration to the type and length of casing to be used, and how it is installed. Steel casing will generally reduce the side resistance of a shaft. No specific data is available regarding the reduction in skin friction resulting from the use of permanent casing relative to concrete placed directly against the soil. Side resistance reduction factors for driven steel piles relative to concrete piles can vary from 50 to 75 percent, depending on whether the steel is clean or rusty, respectively (**Potyondy, 1961**). Greater reduction in the side resistance may be needed if oversized cutting shoes or splicing rings are used.

If open-ended pipe piles are driven full depth with an impact hammer before soil inside the pile is removed, and left as a permanent casing, driven pile static analysis methods may be used to estimate the side resistance as described in **WSDOT GDM Section 8.12.4.7.5**.

The following portion of a drilled shaft, illustrated in **Figure 8-42**, should not be taken to contribute to the development of resistance through skin friction:

- At least the top 5.0 FT of any shaft;
- For straight shafts, a bottom length of the shaft taken as the shaft diameter;
- Periphery of belled ends, if used; and
- Distance above a belled end taken as equal to the shaft diameter.

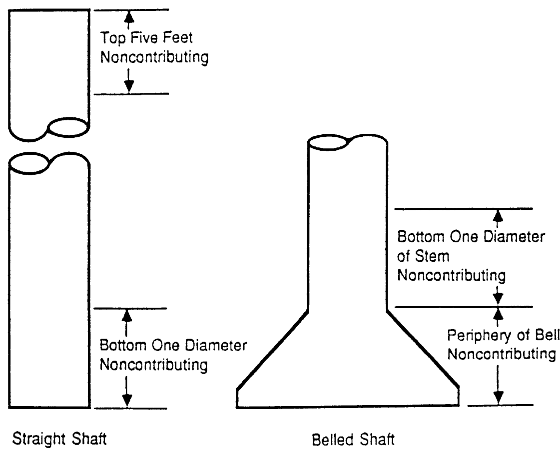


Figure 8-42 Explanation of Portions of Drilled Shafts Not Considered in Computing Side Resistance (O'Neill & Reese, 1999).

The depth of 5.0 FT at the top of shaft may need to be increased if the drilled shaft is installed in expansive clay, if scour deeper than 5.0 FT is anticipated, if there is substantial ground line deflection from lateral loading, or if there are other long-term loads or construction factors that so indicate.

The upper 5.0 FT of the shaft is ignored in estimating R_n, to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete. Regarding the shaft length near the tip, a reduction in the effective length of the shaft contributing to side resistance has been attributed to horizontal stress relief in the region of the shaft tip, arising from development of outward radial stresses at the toe during mobilization of tip resistance. The influence of this effect may extend for a distance of 1B above the tip (**O'Neill & Reese, 1999**). The effectiveness of enlarged bases is limited when L/B is greater than 25 due to the lack of load transfer to the tip of the shaft.

Bells or underreams constructed in stiff fissured clay often settle sufficiently to result in the formation of a gap above the bell that will eventually be filled by slumping soil. Slumping will tend to loosen the soil immediately above the bell and decrease the side resistance along the lower portion of the shaft.

8.13.4.4.1(b) Tip Resistance in Cohesive Soils

For axially loaded shafts in cohesive soil, the nominal unit tip resistance, q_p, by the total stress method as provided in **O'Neill and Reese (1999)** shall be taken in KSF as:

$$q_p = N_c S_u \leq 80.0 \quad (8\text{-}82)$$

in which:

$$N_c = 6\left[1 + 0.2(Z / B)\right] \leq 9 \quad (8\text{-}83)$$

where:

B = diameter of drilled shaft (FT)

Z = penetration of shaft (FT)

S_u = undrained shear strength (KSF)

The value of S_u should be determined from the results of in-situ and/or laboratory testing of undisturbed samples obtained within a depth of 2.0 diameters below the tip of the shaft. If the soil within 2.0 diameters of the tip has S_u <0. 5 KSF, the value of N_c should be multiplied by 0.67.

These equations are for total stress analysis. For effective stress methods for shafts in clay, see **O'Neill and Reese (1999)**.

The limiting value of 80.0 KSF for q_p is not a theoretical limit but a limit based on the largest measured values. A higher limiting value may be used if based on the results of a load test, or previous successful experience in similar soils.

8.13.4.4.2 Estimation of Drilled Shaft Resistance in Cohesionless Soils

Shafts in cohesionless soils should be designed by effective stress methods for drained loading conditions or by empirical methods based on in-situ test results. The factored resistance should be determined using available experience with similar conditions. Although many field load tests have been performed on drilled shafts in clays, very few have been performed on drilled shafts in sands. The shear strength of cohesionless soils can be characterized by an angle of internal friction, ϕ, or empirically related to its SPT blow count, N. Methods of estimating shaft resistance and end bearing are presented below. Judgment and experience should always be considered.

8.13.4.4.2(a) Side Resistance in Cohesionless Soils

The nominal axial resistance of drilled shafts, q_S (KSF), in cohesionless soils by the β-method shall be taken as:

$$q_S = \beta\sigma'_V \leq 4.0 \text{ for } 0.25 \leq \beta \leq 1.2 \quad (8\text{-}84)$$

in which:

$$\beta = 1.5 - 0.135\sqrt{z} \quad \text{for } N_{60} \geq 15 \tag{8-85}$$

or:

$$\beta = \frac{N_{60}}{15}(1.5 - 0.135\sqrt{z}) \quad \text{for } N_{60} < 15 \tag{8-86}$$

where:

σ'_v	=	vertical effective stress at soil layer mid-depth (KSF)
β	=	load transfer coefficient (DIM)
z	=	depth below ground at soil layer mid-depth (FT)
N_{60}	=	average SPT blow count (corrected only for hammer efficiency) in the design zone under consideration (blows/FT)

Higher values may be used if verified by load tests.

O'Neill and Reese (1999) provide additional discussion of computation of shaft side resistance and recommend allowing β to increase to 1.8 in gravels and gravelly sands, however, they recommend limiting the unit side resistance to 4.0 KSF in all soils.

O'Neill and Reese (1999) also provide for computing β in gravelly sands and gravels using **Equation 8-87** where $N_{60} \geq 15$. If $N_{60} < 15$, **Equation 8-86** should be used.

$$\beta = 2.0 - 0.06(z)^{0.75} \tag{8-87}$$

O'Neill and Reese (1999) proposed a method for uncemented soils that uses a different approach in that the shaft resistance is independent of the soil friction angle or the SPT blow count. According to their findings, the friction angle approaches a common value due to high shearing strains in the sand caused by stress relief during drilling.

When permanent casing is used, the side resistance shall be adjusted with consideration to the type and length of casing to be used, and how it is installed. Steel casing will generally reduce the side resistance of a shaft. No specific data is available regarding the reduction in skin friction resulting from the use of permanent casing relative concrete placed directly against the soil. Side resistance reduction factors for driven steel piles relative to concrete piles can vary from 50 to 75 percent, depending on whether the steel is clean or rusty, respectively (**Potyondy, 1961**). Casing reduction factors of 0.6 to 0.75 are commonly used. Greater reduction in the side resistance may be needed if oversized cutting shoes or splicing rings are used.

If open-ended pipe piles are driven full depth with an impact hammer before soil inside the pile is removed, and left as a permanent casing, driven pile static analysis methods may be used to estimate the side resistance as described in **WSDOT GDM Section 8.12.4.7.5**.

8.13.4.4.2(b)Tip Resistance in Cohesionless Soils

The nominal tip resistance, q_p, in KSF, for drilled shafts in cohesionless soils by the O'Neill and Reese method (**O'Neill and Reese, 1999)** shall be taken as:

$$q_p = 1.2N_{60} \quad \text{for } N_{60} \leq 50 \qquad (8\text{-}88)$$

where:

N_{60} = average SPT blow count (corrected only for hammer efficiency) in the design zone under consideration (blows/FT)

Equation 8-88 should be limited to 60 KSF, unless greater values can be justified though load test data.

For cohesionless soil classified as an IGM per **WSDOT GDM Section 8.13.3.1.2** (e.g., cohesionless soils with SPT-N blow counts greater than 50 blows/ft), when using the **O'Neill and Reese (1999)** method, the tip resistance, in KSF shall be taken as:

$$q_p = 0.59\left[N_{60}\left(\frac{p_a}{\sigma'_v}\right)\right]^{0.8}\sigma'_v \qquad (8\text{-}89)$$

where:

p_a = atmospheric pressure (= 2.12 KSF)

σ'_v = vertical effective stress at the tip elevation of the shaft (KSF)

N_{60} should be limited to 100 blows/ft in **Equation 8-89** if higher values are measured.

8.13.4.4.3 Shafts in Strong Soil Overlying Weaker Compressible Soil

Where a shaft is tipped in a strong soil layer overlying a weaker layer, the base resistance shall be reduced if the shaft base is within a distance of 1.5B of the top of the weaker layer. A weighted average should be used that varies linearly from the full base resistance in the overlying strong layer at a distance of 1.5B above the top of the weaker layer to the base resistance of the weaker layer at the top of the weaker layer.

The distance of 1.5B represents the zone of influence for general bearing capacity failure based on bearing capacity theory for deep foundations.

8.13.4.4.4 Estimation of Drilled Shaft Resistance in Rock

Drilled shafts in rock subject to compressive loading shall be designed to support factored loads in:

- Side-wall shear comprising skin friction on the wall of the rock socket; or
- End bearing on the material below the tip of the drilled shaft; or
- A combination of both.

The difference in the deformation required to mobilize skin friction in soil and rock versus what is required to mobilize end bearing shall be considered when estimating axial compressive resistance of shafts embedded in rock. Where end bearing in rock is used as part of the axial compressive resistance in the design, the contribution of skin friction in the rock shall be reduced to account for the loss of skin friction that occurs once the shear deformation along the shaft sides is greater than the peak rock shear deformation (i.e., once the rock shear strength begins to drop to a residual value).

Methods presented in this article to calculate drilled shaft axial resistance require an estimate of the uniaxial compressive strength of rock core. Unless the rock is massive, the strength of the rock mass is most frequently controlled by the discontinuities (including orientation, length, and roughness) and the behavior of the material that may be present within the discontinuity (e.g., gouge or infilling). The methods presented are semi-empirical and are based on load test data and site-specific correlations between measured resistance and rock core strength.

Design based on side-wall shear alone should be considered for cases in which the base of the drilled hole cannot be cleaned and inspected or where it is determined that large movements of the shaft would be required to mobilize resistance in end bearing. Design based on end-bearing alone should be considered where sound bedrock underlies low strength overburden materials (including highly weathered rock). In these cases, however, it may still be necessary to socket the shaft into rock to provide lateral stability. Where the shaft is drilled some depth into sound rock, a combination of sidewall shear and end bearing can be assumed (**Kulhawy and Goodman, 1980**).

If the rock is degradable, use of special construction procedures, larger socket dimensions, or reduced socket resistance should be considered.

For drilled shafts installed in karstic formations, exploratory borings should be advanced at each drilled shaft location to identify potential cavities. Layers of compressible weak rock along the length of a rock socket and within approximately three socket diameters or more below the base of a drilled shaft may reduce the resistance of the shaft.

For rock that is stronger than concrete, the concrete shear strength will control the available side friction, and the strong rock will have a higher stiffness, allowing significant end bearing to be mobilized before the side wall shear strength reaches its peak value. Note that concrete typically reaches its peak shear strength at about 250 to 400 microstrain (for a 10 ft long rock socket, this is approximately 0.5 inches of deformation at the top of the rock socket). If strains or deformations greater than the value at the peak shear stress are anticipated to mobilize the desired end bearing in the rock, a residual value for the skin friction can still be used. **WSDOT GDM Section 8.13.4.4.4(c)** provides procedures for computing a residual value of the skin friction based on the properties of the rock and shaft.

8.13.4.4.4(a) Side Resistance in Rock

For drilled shafts socketed into rock, shaft resistance may be evaluated as follows (**Horvath and Kenney, 1979**):

$$q_s = 0.65\alpha_E p_a (q_u/p_a)^{0.5} < 0.65 p_a (f'_c/p_a)^{0.5} \qquad (8\text{-}90)$$

where:

q_u = uniaxial compressive strength of rock (KSF)

p_a = atmospheric pressure (=2.12 KSF)

α_E = reduction factor to account for jointing in rock as provided in **Table 8-22**

f'_c = concrete compressive strength (KSF)

E_M/E_i	α_E
1.0	1.0
0.5	0.8
0.3	0.7
0.1	0.55
0.05	0.45

Table 8-22 Estimation of α_E (O'Neill and Reese, 1999).

Equation 8-90 applies to the case where the side of the rock socket is considered to be smooth or where the rock is drilled using a drilling slurry. Significant additional shaft resistance may be achieved if the borehole is specified to be artificially roughened by grooving. Methods to account for increased shaft resistance due to borehole roughness is provided in Section 11 of **O'Neill and Reese (1999)**.

Equation 8-82 should only be used for intact rock. When the rock is highly jointed, the calculated q_s should be reduced to arrive at a final value for design. The procedure is as follows:

Step 1. Evaluate the ratio of rock mass modulus to intact rock modulus (i.e., E_m/E_i) in accordance with **WSDOT GDM Chapter 5**.

Step 2. Evaluate the reduction factor, α_E, using **Table 8-22**.

Step 3. Calculate q_s according to **Equation 8-90**.

8.13.4.4.4(b)Tip Resistance in Rock

End-bearing for drilled shafts may be evaluated as follows:

- If the rock below the base of the drilled shaft (to a depth of 2.0 B) is either intact or tightly jointed (i.e., no compressible material or gouge-filled seams) and the depth of the socket is greater than 1.5B (**O'Neill and Reese, 1999**):

$$q_p = 2.5\, q_u \qquad (8\text{-}91)$$

- If the rock below the base of the shaft to a depth of 2.0 B is jointed, the joints have random orientation, and the condition of the joints can be evaluated (**O'Neill and Reese, 1999**):

$$q_p = \left[\sqrt{s} + \sqrt{(m\sqrt{s} + s)}\right] q_u \qquad (8\text{-}92)$$

where:

s, m = fractured rock mass parameters and are defined by reference in **WSDOT GDM Chapter 5**

If end bearing in the rock is to be relied upon, and wet construction methods are used, bottom clean-out procedures such as airlifts should be specified to ensure removal of loose material before concrete placement.

The use of **Equation 8-91** also requires that there are no solution cavities or voids below the base of the drilled shaft.

Equation 8-92 is a lower bound solution for bearing resistance for a drilled shaft bearing on or in (i.e., socketed) a fractured rock mass. This method is appropriate for rock with joints that are not necessarily oriented preferentially and the joints may be open, closed, or filled with weathered material. Load testing will likely indicate higher tip resistance than that calculated using **Equation 8-92**. Resistance factors for this method have not been developed and must therefore be estimated by the geotechnical designer.

8.13.4.4.4(c) Combined Side and Tip Resistance in Rock

Design methods that consider the difference in shaft movement required to mobilize skin friction in rock versus what is required to mobilize end bearing, such as the methodology provided by **O'Neill and Reese (1999)**, shall be used to estimate axial compressive resistance of shafts embedded in rock.

Typically, the axial compression load on a shaft socketed into rock is carried solely in shaft side resistance until a total shaft movement on the order of 0.4 IN occurs.

Designs which consider combined effects of side friction and end-bearing of a drilled shaft in rock require that side friction resistance and end bearing resistance be evaluated at a common value of axial displacement, since maximum values of side friction and end-bearing are not generally mobilized at the same displacement.

Where combined side friction and end-bearing in rock is considered, the geotechnical designer needs to evaluate whether a significant reduction in shaft resistance will occur after the peak shaft resistance is mobilized. As indicated in **Figure 8-43**, when the rock is brittle in shear, much shaft resistance will be lost as vertical movement increases to the value required to develop the full value of q_p. If the rock is ductile in shear (i.e., deflection softening does not occur), then the shaft resistance and end-bearing resistance can be added together directly. If the rock is brittle, however, adding them directly may be unconservative. Load testing or laboratory shear strength testing (e.g., direct shear testing) may be used to evaluate whether the rock is brittle or ductile in shear.

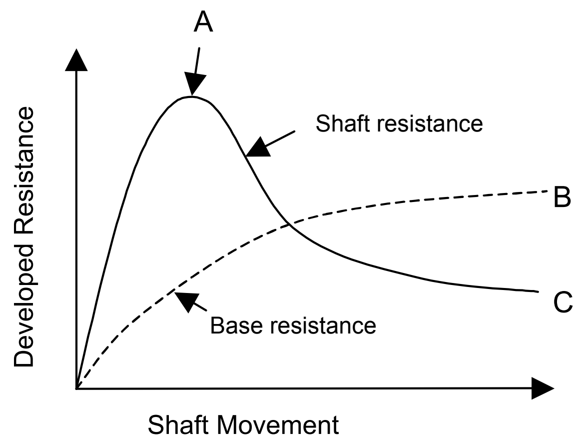


Figure 8-43 Deflection Softening Behavior of Drilled Shafts under Compression Loading (after O'Neill and Reese, 1999).

The method used to evaluate combined side friction and end-bearing at the strength limit state requires the construction of a load-vertical deformation curve. To accomplish this, calculate the total load acting at the head of the drilled shaft, Q_{T1}, and vertical movement, w_{T1}, when the nominal shaft side resistance (Point A on **Figure 8-43**) is mobilized. At this point, some end bearing is also mobilized. For detailed computational procedures for estimating shaft resistance in rock, considering the combination of side and tip resistance, see **O'Neill and Reese (1999)**.

8.13.4.4.5 Estimation of Drilled Shaft Resistance in Intermediate Geomaterials (IGM's)

For detailed base and side resistance estimation procedures for shafts in IGM's, the procedures provided by **O'Neill and Reese (1999)** should be used. See **WSDOT GDM Section 8.13.3.1.2** for a definition of an IGM.

For convenience, since a common situation is to tip the shaft in a cohesionless IGM, the equation for tip resistance in a cohesionless IGM is provided in **WSDOT GDM Section 8.13.4.4.2(b)**.

8.13.4.4.6 Estimation of Drilled Shaft Resistance Using Load Tests

When used, load tests shall be conducted in representative soil conditions using shafts constructed in a manner and of dimensions and materials similar to those planned for the production shafts. The load test shall follow the procedures specified in ASTM D1143. The loading procedure should follow the Quick Load Test Method, unless detailed longer-term load-settlement data is needed, in which case the standard loading procedure should be used.

The nominal resistance shall be determined according to the failure definition of either:

- "plunging" of the drilled shaft, or
- a gross settlement or uplift of 5 percent of the diameter of the shaft if plunging does not occur

Plunging occurs when a steady increase in movement results from incrementally small increases in load (e.g., 2 KIPS).

Load tests should be conducted following prescribed written procedures that have been developed from accepted standards and modified, as appropriate, for the conditions at the site. The Quick Test Procedure is desirable because it avoids problems that frequently arise when performing a static test that cannot be started and completed within an eight-hour period. Tests that extend over a longer period are difficult to perform due to the limited number of experienced personnel that are usually available. The Quick Test has proven to be easily performed in the field, and the results usually are satisfactory. However, if the formation in which the shaft is installed may be subject to significant creep settlement, alternative procedures provided in ASTM D1143 should be considered.

The resistance factors for axial compressive resistance or axial uplift resistance shall be taken as specified in **Table 8-11**.

Regarding the use of shaft load test data to determine shaft resistance, the load test results should be applied to production shafts that are not load tested by matching the static resistance prediction to the load test results. The calibrated static analysis method should then be applied to adjacent locations within the site (see **WSDOT GDM Section 8.9.2** and associated commentary for the definition of a site and number of load tests required to account for site variability) to determine the shaft tip elevation required, in consideration of variations in the geologic stratigraphy and design properties at each production shaft location.

The results of full-scale load tests can differ even for apparently similar ground conditions. Therefore, care should be exercised in generalizing and extrapolating the test results to other locations.

For large diameter shafts, where conventional reaction frames become unmanageably large, load testing using Osterberg load cells may be considered. Additional discussion regarding load tests is provided in **O'Neill and Reese (1999)**. Alternatively, smaller diameter shafts may be load tested to represent the larger diameter shafts (but no less than one-half the full scale production shaft diameter), provided that appropriate measures are taken to account for potential scale effects when extrapolating the results to the full scale production shafts.

8.13.4.5 Shaft Group Resistance

Reduction in resistance from group effects shall be evaluated.

In addition to the overlap effects discussed below, drilling of a hole for a shaft less than three shaft diameters from an existing shaft reduces the effective stresses against both the side and base of the existing shaft. As a result, the capacities of individual drilled shafts within a group tend to be less than the corresponding capacities of isolated shafts. If casing is advanced in front of the excavation heading, this reduction need not be made.

For shaft groups that are collectively tipped within a strong soil layer overlying a soft, cohesive layer, block bearing resistance shall be evaluated in accordance with **WSDOT GDM Section 8.12.4.8**.

8.13.4.5.1 Shaft Groups in Cohesive Soil

The provisions of **WSDOT GDM Section 8.12.4.8** shall apply. The resistance factor for the group resistance of an equivalent pier or block failure shall be taken as specified in **Table 8-9** and shall apply where the cap is or is not in contact with the ground. The resistance factors for the group resistance calculated using the sum of the individual drilled shaft resistances are the same as those for the single-drilled shaft resistances.

8.13.4.5.2 Shaft Groups in Cohesionless Soil

Regardless of cap contact with the ground, the individual nominal resistance of each shaft should be reduced by a factor η for an isolated shaft taken as:

- η = 0.65 for a center-to-center spacing of 2.5 diameters,
- η = 1.0 for a center-to-center spacing of 4.0 diameters or more.

For intermediate spacings, the value of η may be determined by linear interpolation.

The bearing resistance of drilled shaft groups in sand is less than the sum of the individual shafts due to overlap of shear zones in the soil between adjacent shafts and loosening of the soil during construction. The recommended reduction factors are based in part on theoretical considerations and on limited load test results. See **O'Neill and Reese (1999)** for additional details and a summary of group load test results. It should be noted that most of the available group load test results were obtained for sands above the water table and for relatively small groups (e.g., groups of 3 to 9 shafts). For larger shaft groups, or for shaft groups of any size below the water table, more conservative values of η should be considered.

8.13.4.6 Shaft Uplift Resistance

Uplift resistance shall be evaluated when upward loads act on the drilled shafts. Drilled shafts subjected to uplift forces shall be investigated for resistance to pull out, for their structural strength, and for the strength of their connection to supported components.

8.13.4.6.1 Uplift Resistance of Single Shafts

The uplift resistance of a single straight-sided drilled shaft should be estimated in a manner similar to that for determining side resistance for drilled shafts in compression, as specified in **WSDOT GDM Section 8.13.4.4**. In determining the uplift resistance of a belled shaft, the side resistance above the bell should conservatively be neglected if the resistance of the bell is considered, and it can be assumed that the bell behaves as an anchor.

The factored nominal uplift resistance of a belled drilled shaft in a cohesive soil, R_R, should be determined as:

$$\mathrm{R}_R = \varphi_{up}\mathrm{R}_n = \varphi_{up}\mathrm{R}_{s\,bell} \qquad (8\text{-}93)$$

in which:

$$R_{s\,bell} = q_{s\,bell} A_u \qquad (8\text{-}94)$$

where:

q_{sbell}	=	N_uS_u (KSF)
A_u	=	$\pi(D_p^2 - D^2)/4$ (FT^2)
N_u	=	uplift adhesion factor (DIM)
D_p	=	diameter of the bell, as specified in **Figure 8-44** (FT)
D_b	=	depth of embedment in the founding layer, as specified in **Figure 8-44** (FT)
D	=	shaft diameter (FT)
S_u	=	undrained shear strength averaged over a distance of 2.0 bell diameters ($2D_p$) above the base (KSF)
φ_{up}	=	resistance factor specified in **Table 8-9**

If the soil above the founding stratum is expansive, S_u should be averaged over the lesser of either $2.0D_p$ above the bottom of the base or over the depth of penetration of the drilled shaft in the founding stratum.

The resistance factors for uplift are lower than those for axial compression. One reason for this is that drilled shafts in tension unload the soil, thus reducing the overburden effective stress and hence the uplift side resistance of the drilled shaft. Empirical justification for uplift resistance factors is discussed in **WSDOT GDM Section 8.9**, and in **Allen (2005)**.

The value of N_u may be assumed to vary linearly from 0.0 at $D_b/D_p = 0.75$ to a value of 8.0 at $D_b/D_p = 2.5$, where D_b is the depth below the founding stratum. The top of the founding stratum should be taken at the base of the zone of seasonal moisture change. The assumed variation of N_u is based on **Yazdanbod et al. (*1987*)**.

This method does not include the uplift resistance contribution due to soil suction and the weight of the shaft.

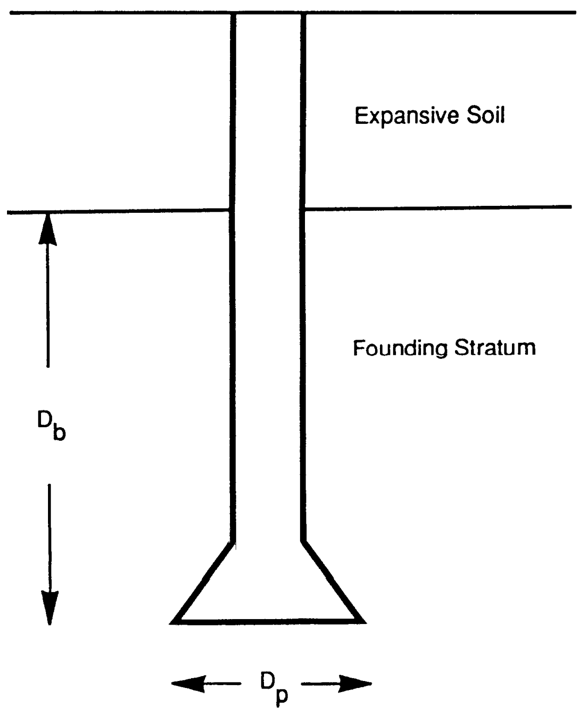


Figure 8-44 Uplift of a belled drilled shaft.

8.13.4.6.2 Uplift Resistance of Shaft Groups

The provisions of **WSDOT GDM Section 8.12.4.10** shall apply.

8.13.4.6.3 Load test for Shaft Uplift Resistance

The provisions of **WSDOT GDM Section 8.12.4.9** shall apply.

8.13.4.7 Nominal Horizontal Resistance of Shaft and Shaft Group Foundations

The provisions of **WSDOT GDM Section 8.12.4.11** shall apply. For shafts classified as long per **Equation 8-95**, P-y methods of analysis may be used. For shafts classified as short or intermediate, when laterally loaded, the shaft maintains a lateral deflection pattern that is close to a straight line. A shaft is defined as short if its length, L, to relative stiffness ratio (L/T) is less than or equal to 2, intermediate when this ratio is less than or equal to 4 but greater than 2, and long when this ratio is greater than 4, where relative stiffness, T, is defined as:

$$T = \left(\frac{EI}{f} \right)^{0.2} \tag{8-95}$$

where,

E = the shaft modulus

I = the moment of inertia for the shaft, and EI is the bending stiffness of the shaft, and

f = coefficient of subgrade reaction for the soil into which the shaft is embedded as provided in **NAVFAC DM 7.2 (1982)**

For shafts classified as short or intermediate as defined above, strain wedge theory **(Norris, 1986; Ashour, et al., 1998)** should be used to estimate the lateral resistance of the shafts.

The design of horizontally loaded drilled shafts shall account for the effects of interaction between the shaft and ground, including the number of shafts in the group. When strain wedge theory is used to assess the lateral load response of shaft groups, group effects shall be addressed through evaluation of the overlap between shear zones formed due to the passive wedge that develops in front of each shaft in the group as lateral deflection increases.

8.13.5 Extreme Event Limit State Design of Drilled Shafts

The provisions of **WSDOT GDM Section 8.12.5** shall apply, except that for liquefaction downdrag, the nominal shaft resistance available to support structure loads plus downdrag shall be estimated by considering only the positive skin and tip resistance below the lowest layer contributing to the downdrag. For this calculation, it shall be assumed that the soil contributing to downdrag does contribute to the overburden stress in the soil below the downdrag zone. In general, the available factored geotechnical resistance should be greater than the factored loads applied to the shaft, including the downdrag, at the strength limit state. The shaft foundation shall be designed to structurally resist the downdrag plus structure loads.

In the instance where it is not possible to obtain adequate geotechnical resistance below the lowest layer contributing to downdrag (e.g., friction shafts) to fully resist the downdrag, the structure should be designed to tolerate the settlement resulting from the downdrag and the other applied loads.

8.14 Micropiles

Micropiles shall be designed using an allowable stress approach until an LRFD approach has been developed and approved by the AASHTO Bridge Subcommittee. The design of micropiles shall be done in accordance with the FHWA Micropile Design and Construction Guidelines Implementation Manual, Publication No. FHWA-SA-97-070 (**Armour, et al., 2000**).

8.15 Proprietary Foundation Systems

Only proprietary foundation systems that have been reviewed and approved by the WSDOT New Products Committee, and subsequently added to **WSDOT GDM Appendix 8-A** of this manual, may be used for structural foundation support.

In general, proprietary foundation systems shall be evaluated based on the following:

1. The design shall rely on published and proven technology, and should be consistent with the AASHTO LRFD Bridge Design Specifications and this geotechnical design manual. Deviations from the AASHTO specifications and this manual necessary to design the foundation system must be fully explained based on sound geotechnical theory and supported empirically through full scale testing.
2. The quality of the foundation system as constructed in the field is verifiable.
3. The foundation system is durable, and through test data it is shown that it will have the necessary design life (usually 75 years or more).
4. The limitations of the foundation system in terms of its applicability, capacity, constructability, and potential impact to adjacent facilities during and after its installation (e.g., vibrations, potential subsurface soil movement, etc.) are clearly identified.

8.16 Detention Vaults

8.16.1 Overview

Requirements for sizing and locating detention/retention vaults are provided in the WSDOT Highway Runoff Manual. Detention/retention vaults as described in this section include wet vaults, combined wet/detention vaults and detention vaults. For specific details regarding the differences between these facilities, please refer to Chapter 5 of the WSDOT Highway Runoff Manual. For geotechnical and structural design purposes, a detention vault is a buried reinforced concrete structure designed to store water and retain soil, with or without a lid. The lid and the associated retaining walls may need to be designed to support a traffic surcharge. The size and shape of the detention vaults can vary. Common vault widths vary from 15 ft to over 60 ft. The length can vary greatly. Detention vaults over a 100 ft in length have been proposed for some projects. The base of the vault may be level or may be sloped from each side toward the center forming a broad V to facilitate sediment removal. Vaults have specific site design elements, such as location with respect to right-of-way, septic tanks and drain fields. The geotechnical designer must address the adequacy of the proposed vault location and provide recommendations for necessary set-back distances from steep slopes or building foundations.

8.16.2 Field Investigation Requirements

A geotechnical reconnaissance and subsurface investigation are critical for the design of all detention vaults. All detention vaults, regardless of their size, will require an investigation of the underlying soil/rock that supports the structure.

The requirements for frequency of explorations provided in **Table 8-23** should be used. Additional explorations may be required depending on the variability in site conditions, vault geometry, and the consequences should a failure occur.

Vault surface area (ft^2)	Exploration points (minimum)
<200	1
200 - 1000	2
1000 – 10,000	3
>10,000	3 - 4

Table 8-23 Minimum exploration requirements for detention vaults.

The depth of the borings will vary depending on the height of soil being retained by the vault and the overall depth of the vault. The borings should be extended to a depth below the bottom elevation of the vault a minimum of 1.5 times the height of the exterior walls. Exploration depth should be great enough to fully penetrate soft highly compressible soils (e.g., peat, organic silt, soft fine grained soils) into competent material of suitable bearing resistance (e.g., very stiff to hard cohesive soil, dense cohesionless soil or bedrock). Since these structures may be subjected to hydrostatic uplift forces, a minimum of one boring must be instrumented with a piezometer to measure seasonal variations in ground water unless the ground water depth is known to be well below the bottom of the vault at all times.

8.16.3 Design Requirements

A detention vault is an enclosed buried structure surrounded by three or more retaining walls. Therefore, for the geotechnical design of detention vault walls, design requirements provided in **WSDOT GDM Chapter 15** are applicable. Since the vault walls typically do not have the ability to deform adequately to allow active earth pressure conditions to develop, at rest conditions should be assumed for the design of the vault walls (see **WSDOT GDM Chapter 15**).

If the seasonal high ground water level is above the base of the vault, the vault shall be designed for the uplift forces that result from the buoyancy of the structure. Uplift forces should be resisted by tie-down anchors or deep foundations in combination with the weight of the structure and overburden material over the structure.

Temporary shoring may be required to allow excavation of the soil necessary to construct the vault. See **WSDOT GDM Chapter 15** for guidelines on temporary shoring. If a shoring wall is used to permanently support the sides of the vault or to provide permanent uplift resistance to buoyant forces, the shoring wall(s) shall be designed as permanent wall(s).

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Appendix 8-A Approved Proprietary Foundation Systems

None.